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Research Project No. 64-6
Lateral Distribution of Load
for Bridges Constructed
with Prestressed Concrete Box Beams

STRUCTURAL BEHAVIOR CHARACTERISTICS of PRESTRESSED CONCRETE BOX-BEAM BRIDGES

by

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LEHIGH UNIVERSITY

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IN PRESTRESSED CONCRETE BOX-BEAM BRIDGES

Reports Completed to Date

LATERAL DISTRIBUTION OF STATIC LOADS IN A PRESTRESSED CONCRETE BOX-BEAM BRIDGE - DREHERSVILLE BRIDGE. Douglas, W. J. and VanHorn, D. A., F. L. Report 315.1, August 1966

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Project No. 64-6: Lateral Distribution of Load for Bridges Constructed with Prestressed Concrete Box Beams

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ABSTRACT

This report summarizes the results from the field-testing of five in-service spread box-beam bridges. The superstructures are of the beam-slab type, and are composed of a number of precast, prestressed concrete box-beams, equally spaced and spread apart, along with a cast-in-place composite slab. The main emphasis is placed on the lateral distribution of vehicular loads, and on beam deflections.

Experimentally-based load distribution factors are compared with values used in the design, and with values derived from a procedure recently recommended by Sanders and Elleby. Based on all facets of this investigation, it is recommended that consideration be given to revision of the current AASHO and PDH procedures for load distribution in spread box-beam bridges. Although new design values are presented, which are based on the field tests, it is emphasized that these experimentally-based values have been superseded by a more extensive analytically-based procedure, as presented in a recent Lehigh University report, No. 315.9.

1. INTRODUCTION

Since the early development of construction techniques for fabrication of prestressed concrete beams in the United States, there have been a number of cross-sectional shapes which have been utilized. One of the shapes which has been used extensively, particularly in the Commonwealth of Pennsylvania, is the box shape. In highway bridge construction, these box-shaped beams were first used in a superstructure design which consisted basically of beams placed adjacent to one another, with longitudinal shear keys to assure lateral interaction. The beams were then covered with an asphalt roadway surface. Later, the adjacent configuration was modified, by replacing the flexible surface material with a cast-in-place reinforced concrete slab, constructed to act compositely with the beams. More recently, the box beams were incorporated into a beam-slab type superstructure design, with the beams spread apart, as in typical I-beam bridges.

To date, the spread box-beam bridges have been designed for vehicular loads, in accordance with standards which closely parallel the procedure outlined in Section 1.3.1 of the AASHO Specifications for Highway Bridges.[1] However, in 1964, a research program was initiated at Lehigh University (1) to develop experimental data which would yield the information on structural behavior needed to evaluate the design procedure, and (2) to develop a mathematical analysis which would accurately represent the structural response to vehicular loads.

In the period 1964-8, five in-service bridges in Pennsylvania were field tested in the investigation. The first bridge, at Drehersville, served as a pilot structure. In addition to the development of basic information on structural behavior, several experimental techniques were evaluated. The second, third, and fourth structures, located at Brookville, Berwick, and White Haven, had nearly identical span lengths, beam spacings, and general cross-sectional dimensions. A comparison of the Brookville (45° skew) and Berwick (90° skew) Bridges revealed the effects of skew, and a comparison of the Berwick (beam size - 48" x 39") and the White Haven (beam size - 36" x 42") Bridges enable determination of the effects of beam width on behavior characteristics. The fifth bridge, near Philadelphia, was used to determine the effects of midspan diaphragms. This bridge was constructed with diaphragms in place. After one series of tests had been completed, the diaphragms were removed, and a second series was conducted. Six reports[2,3,5,6,7,11] have been developed to describe the structural behavior of the individual structures.

A theoretical analysis[12] of the structural response has been completed. The analysis, which is highly complex and therefore not appropriate for direct use in design, has been developed to utilize computer solution techniques. The major factors which influence the lateral distribution of loads have been systematically varied in the program in developing usable expressions for distribution factors. These expressions represent a range of cross-sectional dimensions and beam sizes which are typical of current design standards.

A separate investigation, devoted to a structural model study of the spread box-beam superstructures, was conducted in the period 1965-8. Model beam, slab, curb, and parapet sections were prefabricated in Plexiglas units for the experimental investigation of the effects of several parameters on the lateral distribution of vehicular loads in the spread box-beam bridges. Two reports[4,8] have been developed on this investigation.

This report has been developed (1) to summarize the significant findings which resulted from the field test phase of the investigation, and (2) to recommend the consideration of revisions in the currently used design procedures. A table of suggested distribution factors is presented. To supplement the material and recommendations presented, a comparison of the experimentally-based, proposed distribution factors is made with values developed from expressions presented in the recently completed NCHRP project report by Sanders and Elleby.[10] At this point it should be emphasized that the suggested experimentally-based distribution factors have now been superseded by the more extensive recommendations included in Report No. 315.9.[12]

The material in this report is arranged in the following order. The first two sections are devoted to general descriptions (1) of the cross-section, individual structural elements, and construction of the spread box-beam type superstructure, and (2) of the experimental procedures used in the field tests. The next section is devoted to a presentation of the major results derived from the five field tests, followed by a section centered on discussion of the results. Finally, recommendations for design are presented, based on the results from the field tests.

2. TEST STRUCTURES

2.1 General Description of Superstructure

The test structures were of general beam-slab, simple span construction, utilizing prestressed concrete box-beams as the main longitudinal beams. The general cross-section, along with the main dimensions of the five test structures, is shown in Fig. 1. In this type of construction, after piers and abutments have been completed, the beams are set in place as the first major step in the construction of the superstructure. Forms are then erected for the slab, and for the end and midspan diaphragms between the beams. The slab and diaphragm concrete is then placed, with a finished surface over the roadway portion of the bridge, and a raked finish along the edge portion to provide a construction joint between the slab and the curb sections. In addition to the raked surface, the joint is strengthened by three No. 5 reinforcing bars which extend vertically from the slab up into the curb section, at a spacing not exceeding 15 inches. Two of these bars extend on into the parapet section. The next step involves forming and casting the curb and parapet sections. The curb section is constructed continuously over the length of the slab, while 1/2-inch, open, deflection joints are placed in the parapet section at intervals of approximately 15 feet.

The box beams are pre-cast, prestressed concrete members. A view of the cross-section is shown in Fig. 2. In Pennsylvania, two standard widths (w_b), 36 and 48 inches, are used. The beams are manufactured in a number of depths (h_b), ranging from 21 inches to 48 inches, in 3-inch increments. For all members, the wall thicknesses are 5 inches, the bottom flange thickness is 5 inches, and the top flange thickness is 3 inches. End blocks are utilized in the anchorage zone regions as shown in Fig. 2. In addition, an interior diaphragm, 10 inches in thickness, is located at mid-length of the member.

There are three sets of reinforced concrete diaphragms cast-in-place between the beams in this type of superstructure. All of the diaphragms are cast integrally with the slab. The first set is located at one end of the span, and consists of 15-inch thick sections which extend from the bottom of the slab to the bottom surface of the beams. The second set is located at the other end of the span, and consists of 12-inch thick sections which extend downward 21 inches from the top of the slab. The third set is located at midspan, and consists of 10-inch thick sections which extend from the bottom of the slab to within 9 inches of the bottom surface of the beams.

The reinforced concrete slab is designed essentially as a one-way slab, with main reinforcement in the transverse direction. The transverse reinforcement consists of upper and lower layers of straight bars, with size and spacing of bars identical in both layers. In the longitudinal direction, nominal reinforcement is placed in an upper layer which extends across the entire width of the slab, and a lower layer which extends across the clear spans of the slab between beams.

The general design and construction details for bridges of this type are given in Standards for Prestressed Concrete Bridges.[9]

2.2 Design for Vehicular Loads

All of the test structures were designed essentially in accordance with the AASHO Standard Specifications for Highway Bridges, with some modifications as set forth in the Standards for Prestressed Concrete Bridges.[9] The design highway live loading was HS20-44 in all cases, and the AASHO impact formula was used. The procedures for distributing vehicular loads were generally in line with the provisions of Section 1.3.1 of the AASHO specifications. See Fig. 3. All interior beams were designed utilizing a distribution factor of S/5.5. All fascia beams were designed by assuming simple support action between the exterior and first interior beam, as shown in Fig. 3, with the center of the exterior wheel load located 2 feet from the face of the curb.

3. EXPERIMENTAL PROCEDURE

3.1 Conduct of Tests

The prime, overall objective of the investigation was to evaluate the lateral distribution of vehicular loads to the longitudinal beams. In line with that objective, the general test procedure consisted of driving a test vehicle over the test structure in a set of prescribed lanes, measuring the response of the bridge through use of SR-4 electrical strain gages, and recording the various responses through continuously recording equipment. The main test vehicle used throughout the investigation was a three-axle vehicle, closely simulating the HS20-44 design vehicle. See Fig. 4. With the continuously recording equipment, the basic static effect was evaluated by moving the test vehicle across the structure at crawl speed (approximately 2 mph). In addition, on all structures, passes of the test vehicle were conducted at higher speeds. Although one report[3] was devoted to a description of the effects of vehicular speed on the response of the Drehersville Bridge, the effects of moving loads are not included in this summary report. Reports on structural response as related to the speeds of moving test vehicles on the Philadelphia, Berwick, and White Haven Bridges are being developed by the U. S. Bureau of Public Roads.

3.2 Instrumentation

On all structures, the primary instrumentation consisted of strain gages mounted around the periphery of the longitudinal beams at various cross-sections. In addition, gages were mounted on the curb and parapet sections. Deflection gages were mounted at a section near midspan. The remaining gages were used to measure slab strains and diaphragm strains, and the pattern for these latter gages was varied from structure to structure. The longitudinal locations of the sections at which the primary beam, slab, curb, and parapet gages were located are shown in Fig. 5. For more detailed information on the particular gaging pattern for each individual structure, the reader is referred to the individual reports.[2,5,6,7]

3.3 Data Reduction

Detailed information on data reduction is presented in each of the individual reports. However, it would be appropriate to describe the general philosophy in the derivation of information used to develop distribution factors. The gages were located on the main beams in such a way that the longitudinal strain distribution on each vertical face of the beam could be determined. With these distributions, the internal bending moment in the composite section was determined, based on the assumption that axial force in the members resulting from the vehicular loads was negligible. Since the moduli of elasticity for the beam, slab, curb, and parapet concretes were not known, only moment coefficients were directly developed from the data. Then, the distribution of load to the individual beams was determined by dividing the moment coefficient for each beam by the sum of the moment coefficients for all beams at that cross-section. These distribution coefficients formed the basis for the influence lines presented in Figs. 6-29.

An average value for the modulus of elasticity of the beam concrete in each bridge was derived by equating the total vehicle moment produced across the gaged cross-section in the superstructure, to the sum of the moment coefficients for the individual beams, multiplied by the modulus of elasticity of the beam concrete. This computation was made for each crawl run of the test vehicle. The average values for each bridge, developed from a number of crawl runs, are shown in the table in Fig. 54.

The results from data obtained from other strain and deflection gages were reported individually in the separate reports.

4. RESULTS

4.1 Load Distribution

Since the main objective of the investigation was to evaluate the distribution factors to the individual beams, and to compare the factors developed for the different test structures, sets of influence lines were developed for the individual beams in each of the bridges. See Figs. 6-29. These influence lines reflect the percentage of the total bending moment in each beam at the gaged cross-section, produced by the load vehicle at some specific lateral and longitudinal position on the test structure. The base line of the diagram represents the lateral location of the center of the test vehicle on the bridge roadway, pictured below in each case. The longitudinal location of the test vehicle is shown at the top of the diagram for each case.

To utilize these influence lines to develop experimental distribution factors, two vehicles (for the Drehersville, Berwick, and White Haven Bridges) and three vehicles (for the Philadelphia Bridge) were placed on the roadway in accordance with the lane provisions outlined in Section 1.2.6 of the AASHO specifications. In this regard, the trucks were positioned in the defined lanes so as to produce the maximum moment in the particular beam under consideration.

In Figs. 30-36, a comparison is made between the distribution factors used in the design of the different beams in the four bridges, and the experimentally developed distribution factors derived from the influence lines presented in Figs. 6-29. For instance, in Fig. 30, the experimentally developed distribution factors from two different longitudinal locations of the test vehicle are compared with the design values for each of the three beams in the bridge. The experimental factors were developed from the influence lines given in Figs. 6 and 7. Likewise, Fig. 31 represents a comparison of experimental and design values of distribution factors as derived from the influence lines given in Figs. 8 through 11. Similarly, the comparisons in Figs. 32 through 34 are based on influence lines presented in Figs. 12 through 21, and the comparisons given in Figs. 35 and 36, are derived from influence lines presented in Figs. 22 through 29. The development of the experimental distribution factors given in Figs. 30-36 is given in Table 1.

To facilitate discussion later in this report, a series of figures was prepared to illustrate the effects of slab thickness (t) and the modular ratio between beam concrete material and cast-in-place concrete material (k), on the moment of inertia (I_{na}) and the section modulus (Z_b) of the exterior and interior beams. The idealized composite cross-sections are shown in Fig. 37. In Fig. 38, the variations in I_{na} and Z_b are shown for an interior girder in the Drehersville Bridge. Figs. 39 and 40 represent the I_{na} and Z_b, respectively, for the exterior girder of the Drehersville Bridge. Figs. 41-49 represent similar quantities in the Philadelphia, Berwick, and White Haven Bridges.

4.2 Beam Deflections

To enable an evaluation of the vertical deflection characteristics of the individual beams for the four bridges, four series of influence lines are developed in Figs. 50-53. In these figures, the ordinate represents the vertical deflection of a particular beam, while the base line represents the lateral location of the test vehicle on the roadway. The longitudinal location of the test vehicle is shown in a diagram at the top of each figure. In each of the four cases, the influence lines represent the effect of the load vehicle positioned longitudinally to produce maximum moment in the test structure. The idealized deflection of an entire bridge cross-section is shown in Fig. 54. In the table in this figure, factors are listed which were used to compute an idealized uniform deflection of all beams under the center axle of the vehicle, indicated as Δ in this figure. A comparison of these computed uniform values with actual beam deflections can be made in Figs. 55 and 56. In these figures, beam deflection profiles are shown, as developed from the influence lines for vertical deflection presented in Figs. 50-53. The idealized values presented in Fig. 54 are found to be closely related to the average deflection of the beams as shown in Figs. 55 and 56.

4.3 Other Results

4.3.1 Effect of Midspan Diaphragms

Although a complete report[7] is devoted to the effect of midspan diaphragms on lateral load distribution in one of the test bridges, it would be appropriate to mention the significant findings. First of all, it can be seen from Fig. 31 that there was very little variation in maximum moment produced in the individual beams at the maximum moment section. The results shown were developed both with midspan diaphragms in place, and with midspan diaphragms removed. Therefore, since load distribution factors used in design represent the combined effects of several vehicles, the midspan diaphragms have very little effect on the design factors. However, it should be emphasized that even though there was very little difference in the combined effects, there was a definite difference in the response of the bridge to a single load vehicle. For instance, for a test vehicle located directly over the center beam, the center beam developed 32.5 percent of the total moment, with diaphragms removed. On the other hand, under the same load condition, the center beam carried 27.5 percent of the total moment, with diaphragms in place. Another comparison of the effects of the midspan diaphragm can be made in Fig. 55, which illustrates the deflection profile at the maximum moment section under a particular loading condition, both with diaphragms in place and with diaphragms removed. For a more complete comparison, the reader is referred to the individual report.[7]

4.3.2 Effect of Skew

The fifth bridge tested in this investigation was a 45° skew bridge having cross-sectional characteristics nearly identical to those of the Berwick Bridge. A separate report[11] was devoted to describing the behavior of this particular bridge. Although no further information on the results from the test of the skew bridge is presented in this report, it would be appropriate to mention the significant conclusions. It was found that beam moments measured in the center region of the skew bridge were consistently less than those measured at the maximum moment section in the Berwick Bridge. Likewise, beam deflections measured along the skew at midspan were similarly less than values measured in the Berwick Bridge. Further comparisons and additional information are given in the individual report.

4.3.3 Modulus of Elasticity

As mentioned in Section 3.3, the effective modulus of elasticity of the beam concrete was computed based on data collected from each crawl run of the test vehicle on each of the test bridges. These individually computed values are reported in the separate reports. Average values are given in Fig. 54.

4.3.4 Effective Slab Widths

In the procedure for computing internal beam moments, determinations were made of effective slab widths. Each of the indivivual reports contains information on transformed effective slab widths at the gaged cross-sections.

4.3.5 Behavior of Midspan Diaphragms

Although no detailed analysis was made of midspan diaphragm behavior, several strain gages were used to measure concrete surface strains on the diaphragms in both the Berwick and White Haven Bridges.

4.3.6 Effect of Deflection Joints

To evaluate the effect of the parapet deflection joints on the load distribution at a particular cross-section, both the Berwick and White Haven Bridges were gaged at a cross-section which passed through one of the joints. Moment and distribution coefficients were developed for these sections, and compared with similar values obtained at the section of the maximum moment in each case. The comparisons can be made in the influence lines presented in Figs. 12-17 and Figs. 22-29, and in the experimentally developed distribution factors presented in Figs. 32, 33, 35 and 36. In the latter comparison, it is apparent that the parapet deflection joints have little effect on the distribution factors.

4.3.7 Use of Super-Position in Developing Test Results

In the test of the Drehersville Bridge, two concepts were used to check the accuracy of using the principle of super-position in combining the results obtained from single vehicle runs to reflect the effects of more than one vehicle on the structure. First of all, all five beams were gaged at one cross-section of the superstructure. A single truck was then driven across the structure in each of seven prescribed lanes. Computations of moment and distribution coefficients were made directly, based on data from all gages.

Comparisons were then made utilizing data from only three of the five beams, and using the principle of super-position.

In a second check, two load vehicles were used. Each was driven across the test structure separately, and the computed combined effects were compared with those developed in running the two trucks across the span simultaneously. In general, the results from both methods indicated a very favorable comparison. For more detailed information, the reader is referred to the individual report.[2]

5. DISCUSSION OF RESULTS

5.1 Load Distribution Factors

Based on the comparisons of experimentally developed distribution factors with the calculated design values, as illustrated in Figs. 30-36, a general behavior characteristic is very apparent. Specifically, in all of the test structures, the experimental values for all interior girders are significantly less than design values. Correspondingly, the experimental values for all exterior girders are somewhat larger than design values. This behavior clearly emphasizes the fact that the curb and parapet sections definitely and significantly contribute to the longitudinal flexural stiffness of the bridge superstructure.

The strain measurements on the curb and parapet sections, when aligned with those on the slab and exterior beam, consistently indicate a full composite behavior between the exterior beam, slab, and curb sections. In addition, the full composite behavior extended to the top of the parapet section in three of the bridges. In the fourth bridge, although the straight line behavior did not extend through the parapet, there was partial participation. The full participation of the curb section was not unexpected, primarily because the curb is continuous over the entire length of the structure, and because the construction joint was sufficient to provide complete composite action at the curb-slab interface. However, the participation of the parapet sections was not as consistent, mainly due to the incidence of the discontinuities provided by the deflection joints.

With the additional stiffness resulting from the interaction of the curb and parapet sections with the slab, the exterior girder consistently carries more load than is computed in the design method. With the increase in load carried by the exterior girders, the interior girders then carry less of the total load, and the maximum magnitudes are considerably less than the design load values. This curb-parapet effect was particularly significant in the three 2-lane bridges (Drehersville, Berwick, and White Haven). In the 3-lane bridge (Philadelphia) the effect was slightly reduced, which is not surprising, since the sizes of the curb and parapet sections were the same in all test structures. Therefore, with the greater width of the Philadelphia Bridge, the contribution of curb and parapet sections to total superstructure stiffness was less than the contributions in the three narrower 2-lane bridges. It is obvious that the contribution would continue to diminish with increasing roadway width.

At this point, it is appropriate to point out that although the experimental results indicated maximum loads greater than design values for exterior beams, the exterior beams were definitely not overstressed. The participation of the curb and parapet sections served to increase the flexural stiffness to the extent that the maximum flexural stresses produced in the exterior girders were proportionately reduced, generally to a level approximately equal to, or below, the maximum values produced in the interior girders.

5.2 Beam Deflections

In comparing measured deflections with the computed values, which were based on the idealized assumption that the deflection is the same for all beams, several points should be noted. First of all, the computed values given in Fig. 54 are based on values of E measured in the load distribution phase of the study, and on a moment of inertia for the entire cross-section of the superstructure. For these computations, (1) the moment of inertia was taken as the sum of the individual values for each of the interior and exterior beams, (2) a modular ratio of 0.7 was used for all bridges, and (3) the curb and parapet sections were assumed to be fully effective. For the Drehersville and White Haven Bridges, the computed values closely approximate the average deflection of the beams, as indicated in Figs. 55 and 56. For the Berwick Bridge, the computed value is slightly less than the average deflection measured experimentally while in the Philadelphia Bridge, the computed value is slightly greater than the measured average. It is certain that the parapet was not fully effective over the entire length of the structures. If this factor could be taken into account, all computed values would be slightly greater than the values indicated in Fig. 54. However, if the computed values had been based on (1) values of E computed as a function of f'_c in currently used expressions, and (2) values of I based on the assumption that the curb and parapet sections contribute nothing to the stiffness, the computed values would have been significantly greater than the measured values. These comparisons further substantiate the participation of the curb and parapet in increasing the longitudinal stiffness of the superstructure, and in addition, confirm that the modulus of elasticity of the beam concrete is definitely greater than values of E based on the 28-day f'_c assumed in design.

5.3 Other Results

5.3.1 Effect of Midspan Diaphragms

The intended function of the midspan diaphragms is to distribute the vehicular loads more uniformly than if the diaphragms were not used. In the Philadelphia Bridge, this effect was quantitatively evaluated. For a single vehicle on the structure, there was a distinct difference in distribution of the load to the individual beams. However, the maximum load conditions for each of the girders always involved the consideration of vehicles in all load lanes. Therefore, the combined effects used to develop the distribution factors resulted in nearly identical values, either with diaphragms in place or removed. This behavior is not surprising, since the superstructure tends to exhibit primarily simple beam behavior with all lanes loaded. Therefore, it is felt that the diaphragms really serve no purpose when the superstructure is subjected to design load conditions. On the other hand, since the midspan diaphragms do serve to more evenly distribute the effects of a single vehicle, the effect of these diaphragms would be far more significant in considering the effects produced when a heavier-than-design vehicle is moved across the structure.

5.3.2 Modulus of Elasticity

It is felt that the E values, calculated as described earlier in Section 3.3, are definitely realistic, even though they are greater than those normally used in design. This is not meant to imply that empirical expressions for E, expressed as a function of f'_c, do not yield reasonable values. Instead, it is felt that the primary difference lies in the value of f'_c used in the calculation of E. That is, the value of f'_c after the superstructure is in use is usually significantly greater than the 28-day value used in design. In fact, it is very common for beam concrete to reach the specified 28-day value at, or shortly after, release of the pre-tensioning elements.

6. RECOMMENDATIONS

6.1 Load Distribution

In initiating a discussion of recommendations relative to vehicular load distribution, it would be appropriate to note that during the process of sponsors' review of this report, an analytical procedure was completed and published as Report No. 315.9[12]. In the development of the analysis, emphasis was placed on correlation of results with the actual behavior of field test structures. Based on all facets of the overall investigation, it is recommended that consideration be given to revision of the procedure for load distribution. Specific recommendations are given in Report No. 315.9. Therefore, it should be noted that these recommendations supersede the recommended distribution factors listed in Table 2 on page 33 of this report, set forth for preliminary consideration prior to the completion of Report No. 315.9.

At this point it would be appropriate to briefly describe a recently distributed report by W. W. Sanders, Jr. and H. A. Elleby.[10] This is a final report entitled Distribution of Wheel Loads on Highway Bridges, resulting from a study sponsored as a part of the National Cooperative Highway Research Program. In this report, significant changes are suggested for determining vehicular load distribution factors to be utilized in design, to replace the current Section 1.3.1 in the AASHO Specifications. First of all, it is suggested that for all beams, both interior and exterior, the same distribution factor would be used for a specific bridge, namely:

$$\text{Distribution Factor} = \frac{S}{D} \qquad (1)$$

where D is expressed as:

$$D = 5 + \frac{L}{10} + \left(3 - \frac{2N_L}{7}\right)\left(1 - \frac{C}{3}\right)^2 \quad \text{for } C \leq 3 \qquad (2)$$

$$D = 5 + \frac{N_L}{10} \quad \text{for } C > 3 \qquad (3)$$

In the above expressions,

- S = average beam spacing, ft.
- N_L = total number of design traffic lanes
- C = a stiffness parameter which depends upon the type of bridge, bridge and beam geometry, and material properties

For preliminary designs, C may be approximated as:

$$C = K \frac{W}{L} \qquad (4)$$

where

- K = 1.8 for spread box-beams
- W = overall width of bridge superstructure, ft.
- L = span length, ft.

A more exact expression for C is given as

$$C = 1.2 \frac{W}{L} \sqrt{\frac{I_b}{b^2 h^2} \left(\frac{h}{t_w} + \frac{b}{\bar{t}}\right)} \cdot \left(1 + \frac{t_w}{b}\right)\left(1 + \frac{\bar{t}}{h}\right) \tag{5}$$

where

I_b = moment of inertia of the beam, in^4.

b = external width of beam, in.

h = external height of beam, in.

t_w = vertical wall thickness, in.

$\bar{t}$ = $\frac{1}{2}(t_t + t_b)$, in.

t_t = thickness of top flange, in.

t_b = thickness of bottom flange, in.

As an indication of the D factors which would be developed from the proposed method, Table 3 has been prepared to indicate the values for D as calculated from the proposed theory.

It should be noted (1) that the expressions proposed by Sanders and Elleby were developed from an analysis which does not include the effects of curb and parapet sections, and (2) that there is little variation in the values of D computed for the different bridges. It can also be seen that the D values are slightly less than the values listed in Table 2. This would be expected since the factors listed in Table 2 reflect behavior resulting from complete interaction of the curb and partial interaction of the parapet with the beam-slab system.

Based on the consistently observed behavior of the field test structures, it is recommended that the construction joint between the curb and the slab be revised to further insure composite interaction between the slab and exterior beam. Under the present construction procedure, this interaction consistently occurs. Even though it is also apparent that the parapet section is at least partially effective, it is felt that it would be conservative and appropriate to ignore this participation in design calculations.

6.2 Deflections

In the computation of deflections for design purposes, it is recommended that consideration be given to the inclusion of the curb and parapet sections as included in the computation of idealized bridge deflections presented in Fig. 54. It is felt that the principle included in these computations will result in a more accurate prediction of the deflection of the bridge in service. If there is a possibility that curb and parapet sections would be removed at a later time to provide for the widening of the structure or for some other purpose, the deflection of the "stripped down" superstructure can easily be calculated by revising the I_{na} values computed for the exterior girders.

In addition, it is also recommended that typical cylinder strength data be used to develop information on beam concrete strengths to evaluate typical strengths during the usable life of the structure. Use of the higher strengths to predict the effective E values will also result in a more accurate computation of deflection.

6.3 Effective Slab Width

Based on the multitude of computations of effective slab width in the many determinations used to determine distribution factors, it is felt that the effective slab width may be accurately assumed equal to the center-to-center spacing of the beams. For the typical dimensions found in the beam slab structures of the spread box beam type, the effective slab width criteria set forth in Section 1.7.99 of the AASHO Specifications[1] indicate that the center-to-center spacing would typically be the governing factor in assuming an effective flange width for design purposes.

7. ACKNOWLEDGMENTS

This study was conducted in the Department of Civil Engineering at Fritz Engineering Laboratory, under the auspices of the Lehigh University Institute of Research, as part of a research investigation sponsored by the Pennsylvania Department of Highways; the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads; and the Reinforced Concrete Research Council.

The field test equipment was made available through Mr. C. F. Scheffey, Chief, Structures and Applied Mechanics Division, Office of Research and Development, Bureau of Public Roads. The instrumentation and operation of the test equipment was managed primarily by Mr. Robert F. Varney, assisted by Mr. Harry Laatz, both from the Bureau of Public Roads.

The basic research planning and administrative coordination in this investigation were in cooperation with the following individuals representing the Pennsylvania Department of Highways: Mr. K. H. Jensen, formerly Bridge Engineer, Mr. B. F. Kotalik, Bridge Engineer, and Mr. H. P. Koretzky, Engineer in Charge of Prestressed Concrete Structures, all from the Bridge Engineering Division; and Mr. Leo D. Sandvig, Director, Mr. Wade L. Gramling, Research Engineer, and Mr. Foster C. Sankey, Research Coordinator, all from the Bureau of Materials, Testing, and Research.

The following Lehigh University graduate students made major contributions in the conduct of the field tests, reduction

and processing of data, and development of the individual reports: Albert A. Guilford, Walter J. Douglas, Thomas Schaffer, and Cheng-shung Lin.

8. TABLES

Table 1 Development of Experimental Distribution Factors

Bridge	Beam	Case	Values from Influence Lines				Distribution Factors	
			Load Lane			Total		
			Left	Center	Right	ΣL+C+R	Experi-mental	Design
Drehersville	A	D-1	40.0	----	12.4	52.4	1.05	0.81
		D-2	40.8	----	11.4	52.2	1.04	
	B	D-1	28.5	----	14.0	42.5	0.85	1.30
		D-2	28.6	----	13.7	42.3	0.85	
	C	D-1	20.0	----	20.0	40.0	0.80	1.30
		D-2	20.5	----	20.5	41.0	0.82	
Philadelphia	A	P-1	46.0	17.1	6.2	69.3	1.39	1.16
		P-2	47.6	14.6	5.8	68.0	1.36	
		P-3	43.0	16.4	6.4	65.8	1.32	
		P-4	44.6	14.2	5.7	64.5	1.29	
	B	P-1	31.5	23.4	10.0	64.9	1.30	1.73
		P-2	34.1	24.0	7.9	66.0	1.32	
		P-3	32.1	22.7	9.8	64.6	1.29	
		P-4	35.0	24.1	8.0	67.1	1.34	
	C	P-1	17.2	27.5	17.2	61.9	1.24	1.73
		P-2	16.9	32.6	16.9	66.4	1.33	
		P-3	17.8	29.4	17.8	65.0	1.30	
		P-4	17.5	32.1	17.5	67.1	1.34	
Berwick	A	B-1	43.8	----	14.3	58.1	1.16	1.05
		B-2	41.8	----	15.8	57.6	1.15	
		B-3	42.6	----	13.1	55.7	1.11	
		B-4	43.9	----	11.6	55.5	1.11	
		B-5	41.8	----	14.4	56.2	1.12	
		B-6	46.7	----	14.0	60.7	1.21	
		B-7	38.5	----	15.1	53.6	1.07	
		B-8	41.8	----	13.1	54.9	1.10	
		B-9	45.1	----	10.4	55.5	1.11	
		B-10	40.9	----	13.0	53.9	1.08	
	B	B-1	31.5	----	20.1	51.6	1.03	1.60
		B-2	33.3	----	18.7	52.0	1.04	
		B-3	35.2	----	20.5	55.7	1.11	
		B-4	40.3	----	19.6	59.9	1.20	
		B-5	31.1	----	22.1	53.2	1.06	
		B-6	32.7	----	18.2	50.9	1.02	
		B-7	31.7	----	21.5	53.2	1.06	
		B-8	34.6	----	21.0	55.6	1.11	
		B-9	40.9	----	19.8	60.7	1.21	
		B-10	33.6	----	21.4	55.0	1.10	

Table 1 (continued)

Bridge	Beam	Case	Values from Influence Lines				Distribution Factors	
			Load Lane			Total ΣL+C+R	Experi-mental	Design
			Left	Center	Right			
White Haven	A	W-1	48.3	----	10.3	58.6	1.17	1.00
		W-2	46.8	----	11.4	58.2	1.16	
		W-3	48.0	----	12.1	60.1	1.20	
		W-4	45.3	----	14.1	59.4	1.19	
		W-5	49.3	----	11.5	60.8	1.22	
		W-6	49.8	----	9.5	59.3	1.19	
		W-7	47.6	----	10.2	57.8	1.16	
		W-8	49.4	----	9.5	58.9	1.18	
	B	W-1	35.4	----	20.0	55.4	1.11	1.64
		W-2	31.0	----	21.3	52.3	1.05	
		W-3	36.8	----	20.4	57.2	1.14	
		W-4	30.3	----	22.0	52.3	1.05	
		W-5	33.5	----	19.0	52.5	1.05	
		W-6	41.8	----	17.2	59.0	1.18	
		W-7	36.3	----	20.1	56.4	1.13	
		W-8	40.4	----	18.8	59.2	1.18	

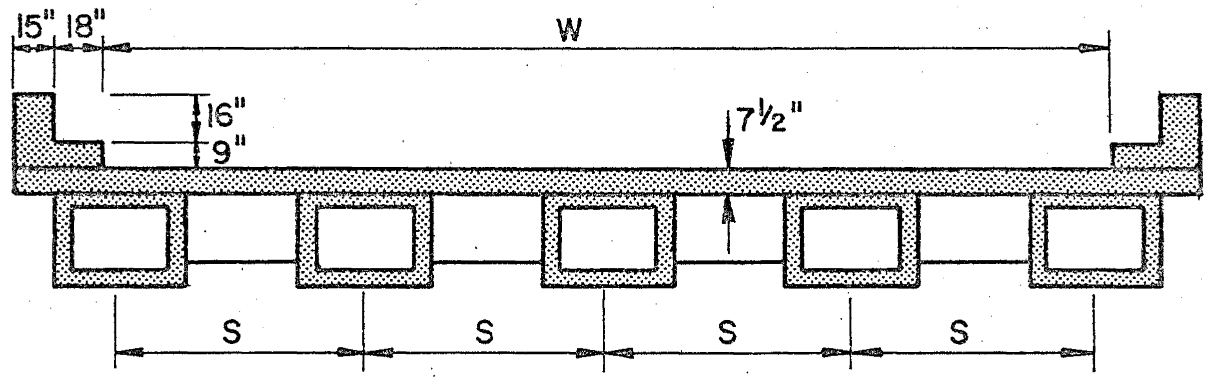
Table 2 Recommended Distribution Factors

	Distribution Factor			
	4-beam		5 beam	
	Exterior	Interior	Exterior	Interior
2-lane	S/6.5	S/6.5	S/6.0	S/7.5
3-lane	-----	-----	S/6.0	S/6.5

Table 3 Values of D (Sanders-Elleby)

Bridge	N_L	W ft.	L ft.	D (Eqs. 2,4)	D (Eqs. 2,5)
Drehersville	2	35.5	61.5	6.35	6.65
Philadelphia	3	45.5	71.8	6.29	6.40
Berwick	2	33.5	65.2	6.49	6.66
White Haven	2	33.5	64.7	6.49	6.50

9. FIGURES



Bridge	Skew	Roadway Width (W)	Beam Size	Beam Spacing (S)	No. of Beams	Beam Span
Drehersville (S-4251A)	90°	30'-0"	4' x 33"	7'-2"	5	61'-6"
Berwick S-5357A)	90°	28'-0"	4' x 39"	8'-9⅜"	4	65'-3"
White Haven (S-5767A)	82°	28'-0"	3' x 42"	9'-0"	4	64'-8"
Brookville (S-4737A)	45°	28'-0"	4' x 36"	8'-10"	4	64'-10½"
Philadelphia (S-6624A)	87°	40'-0"	4' x 42"	9'-6"	5	71'-9"

Fig. 1 Cross-sectional Dimensions of Test Structures

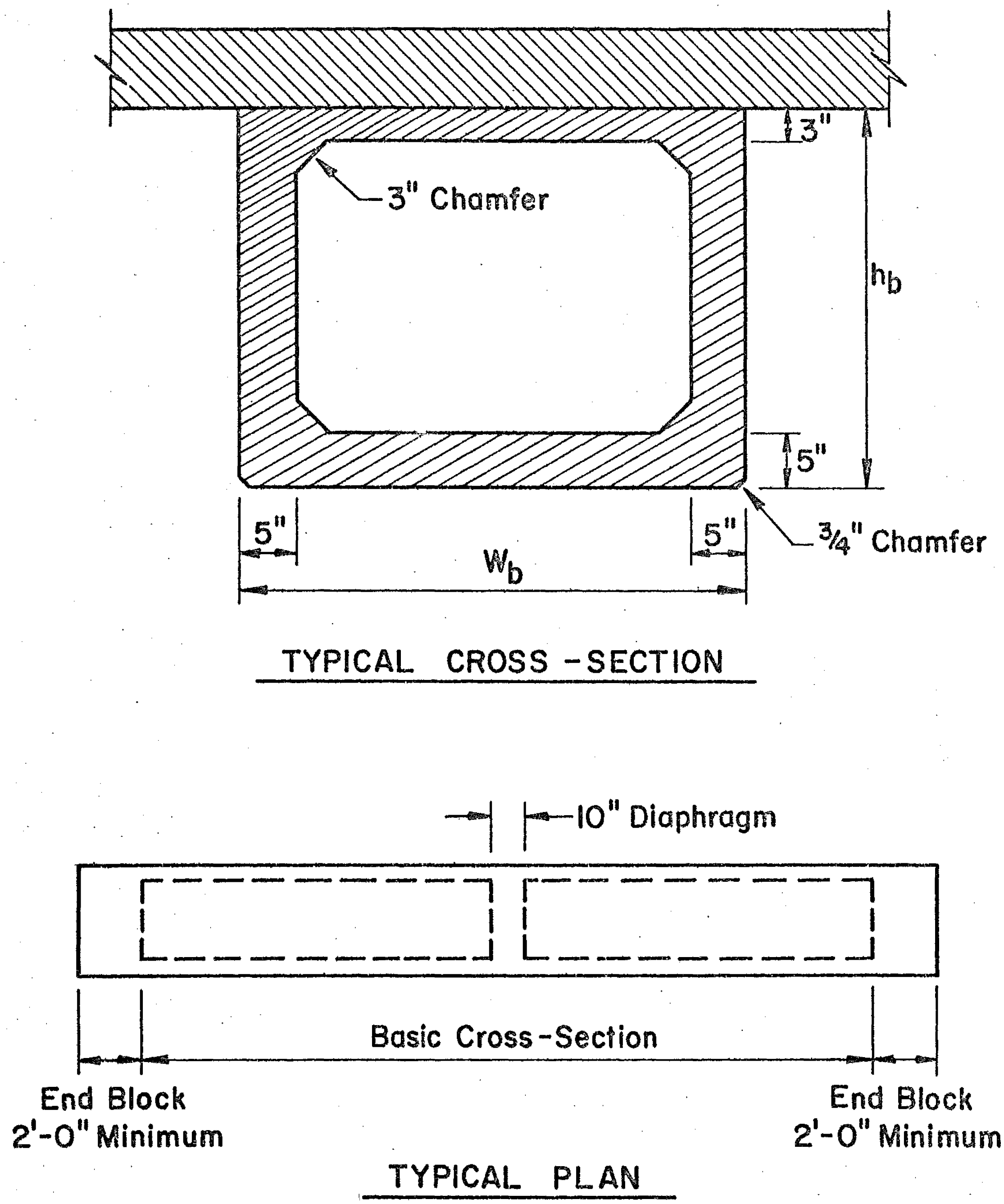


Fig. 2 General Description of Prestressed Concrete Beams

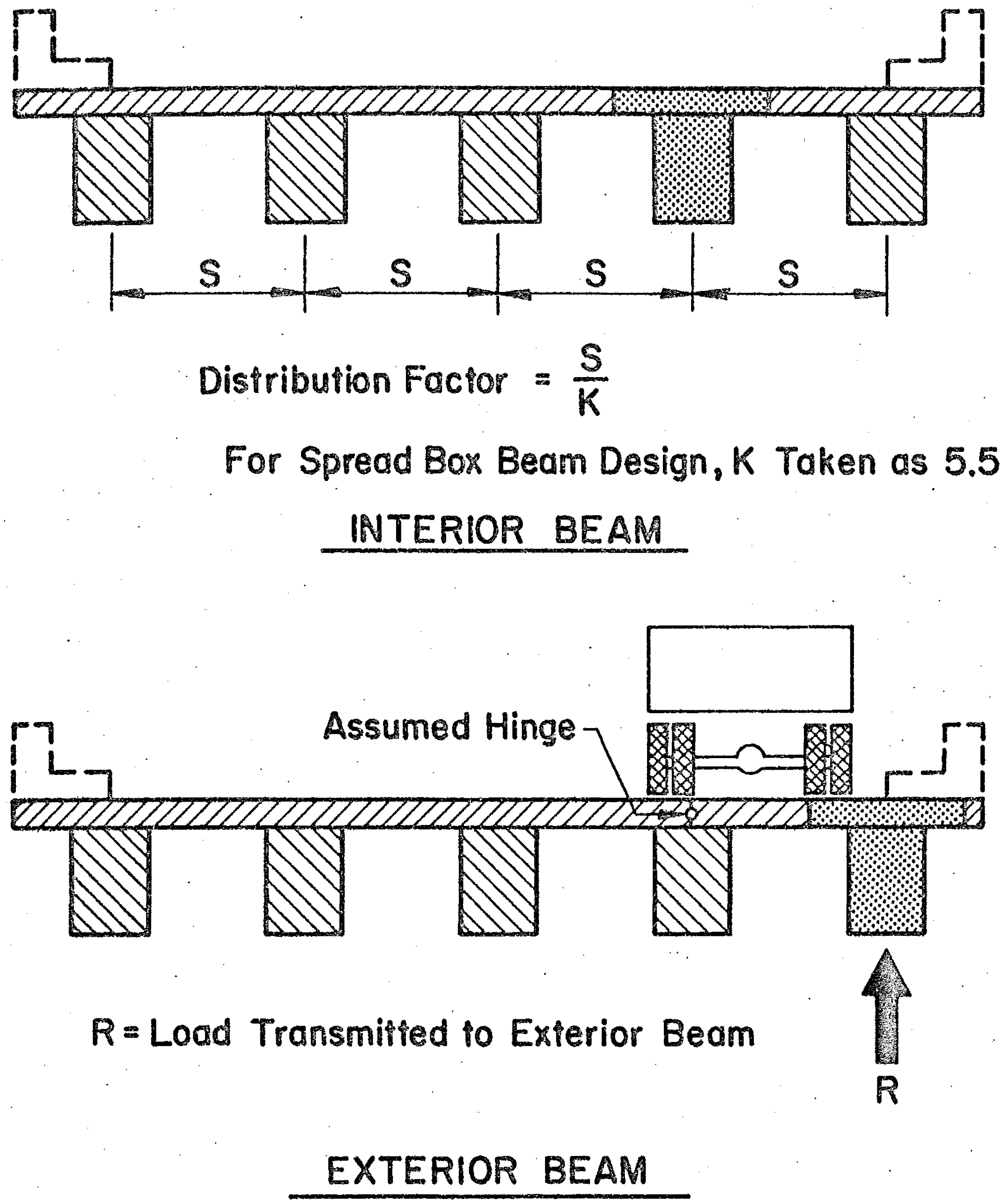
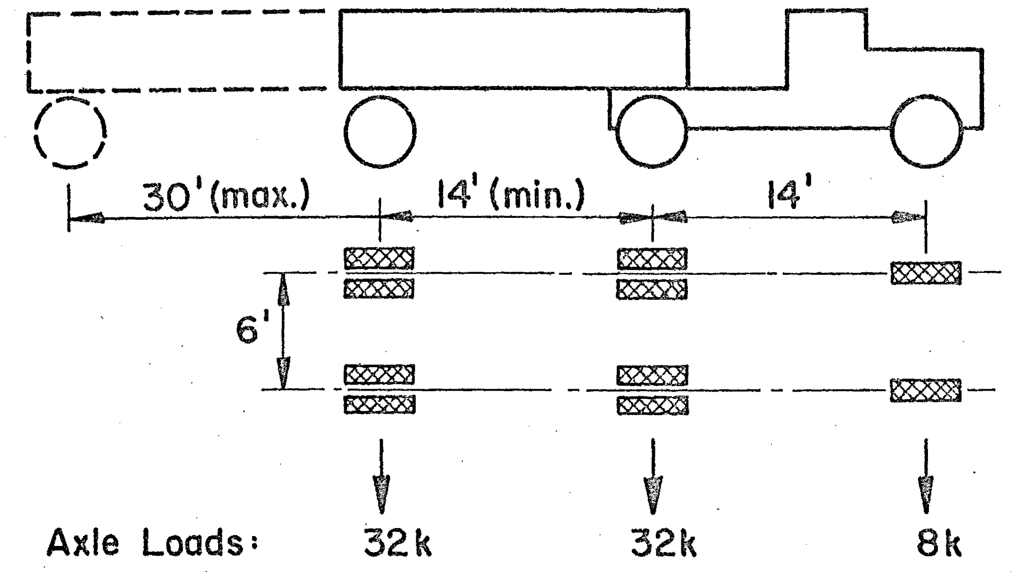
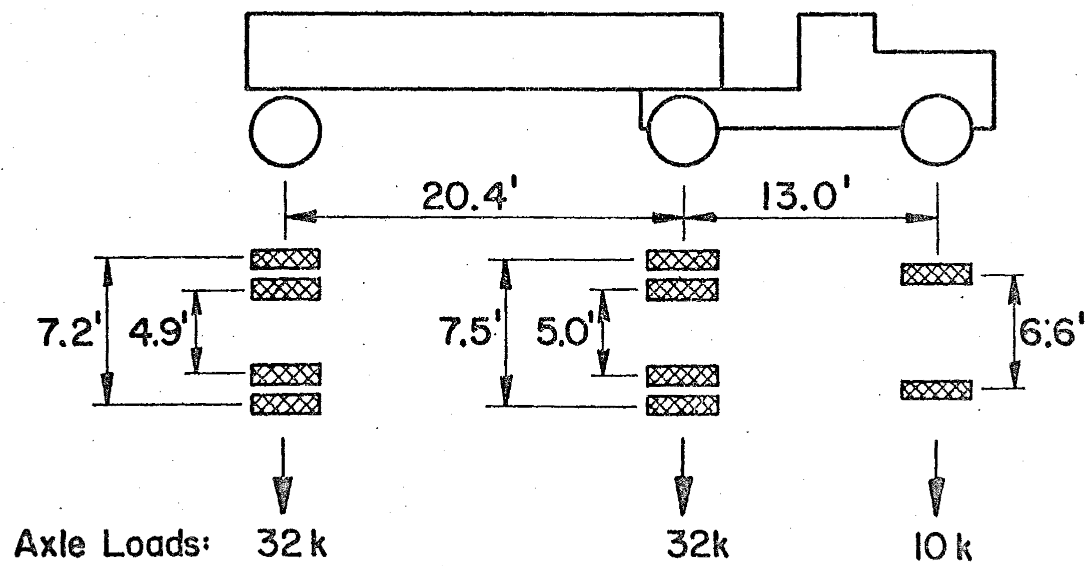


Fig. 3 Distribution of Vehicular Loads Assumed in Design of Spread Box-Beam Superstructures



HS 20-44 Design Load Vehicle



TEST VEHICLE

Fig. 4 Characteristics of Design and Test Vehicles

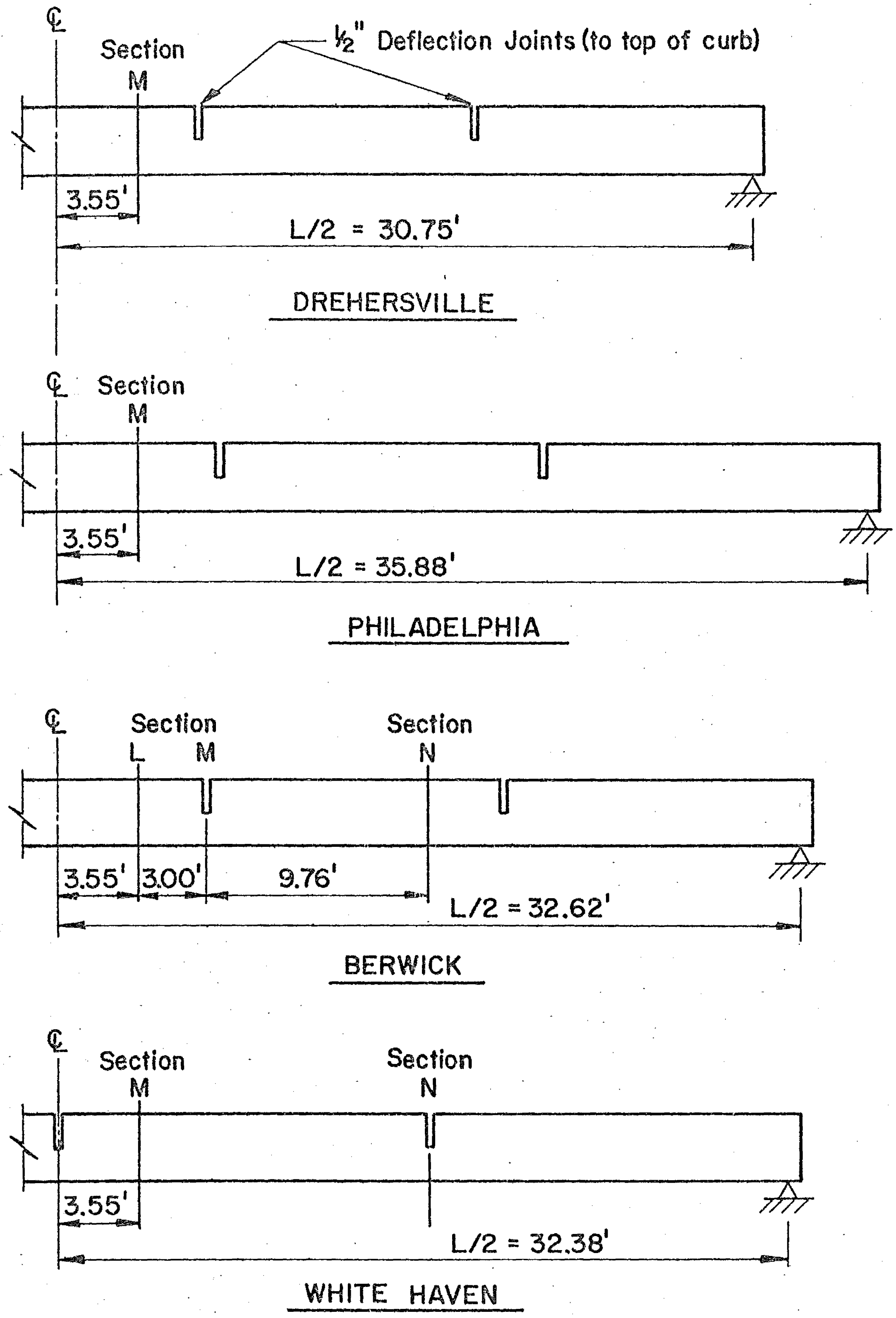


Fig. 5 Locations of Instrumented Cross-sections

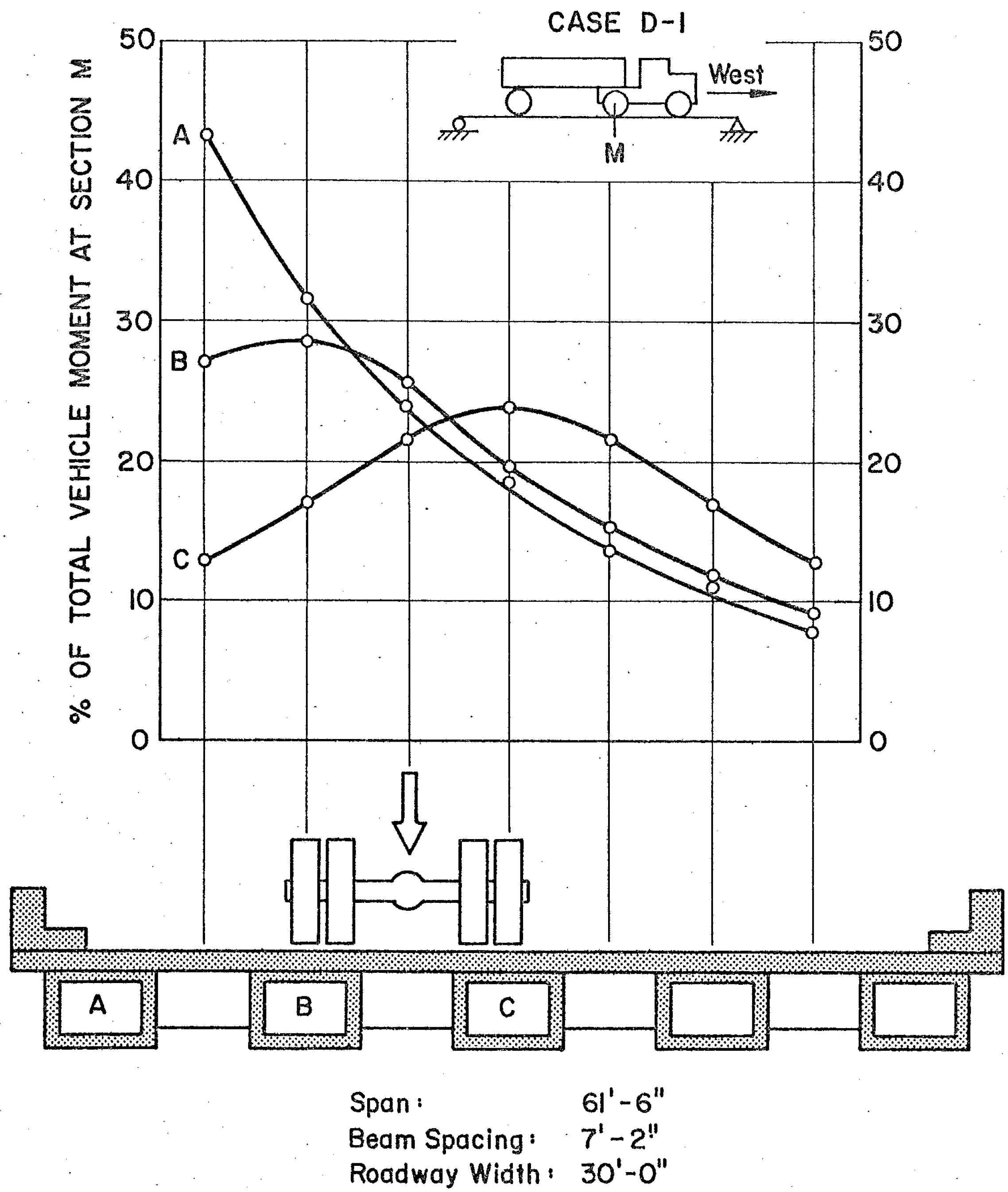
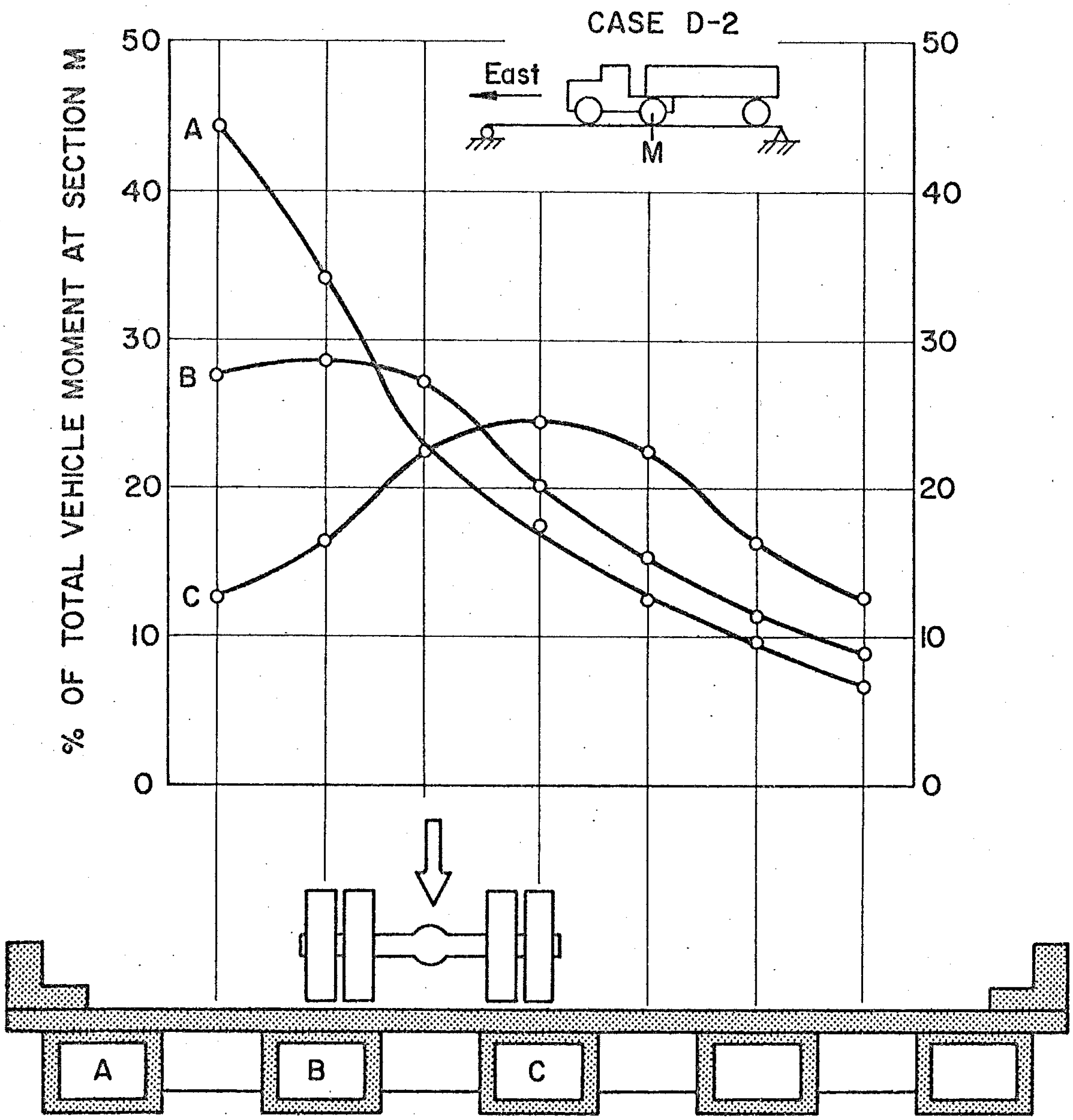
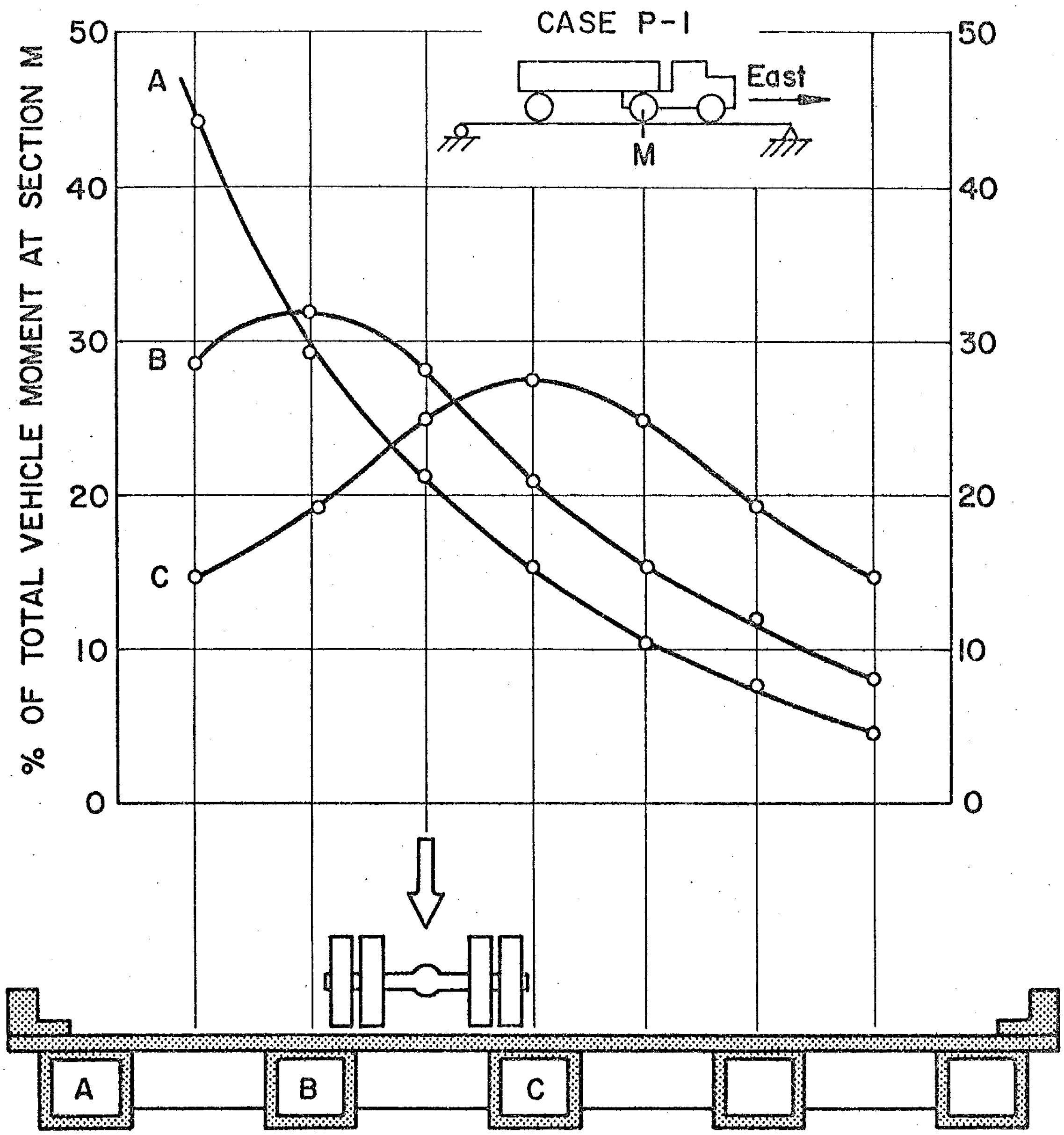


Fig. 6 Influence Lines for Bending Moments in Beams
Drehersville Bridge - Section M - Case D-1



Span: 61'-6"
Beam Spacing: 7'-2"
Roadway Width: 30'-0"

Fig. 7 Influence Lines for Bending Moments in Beams
Drehersville Bridge - Section M - Case D-2



Span: 71'-9"
Beam Spacing: 9'-6"
Roadway Width: 40'-0"

Fig. 8 Influence Lines for Bending Moments in Beams
Philadelphia Bridge (Midspan diaphragms in place)
Section M - Case P-1

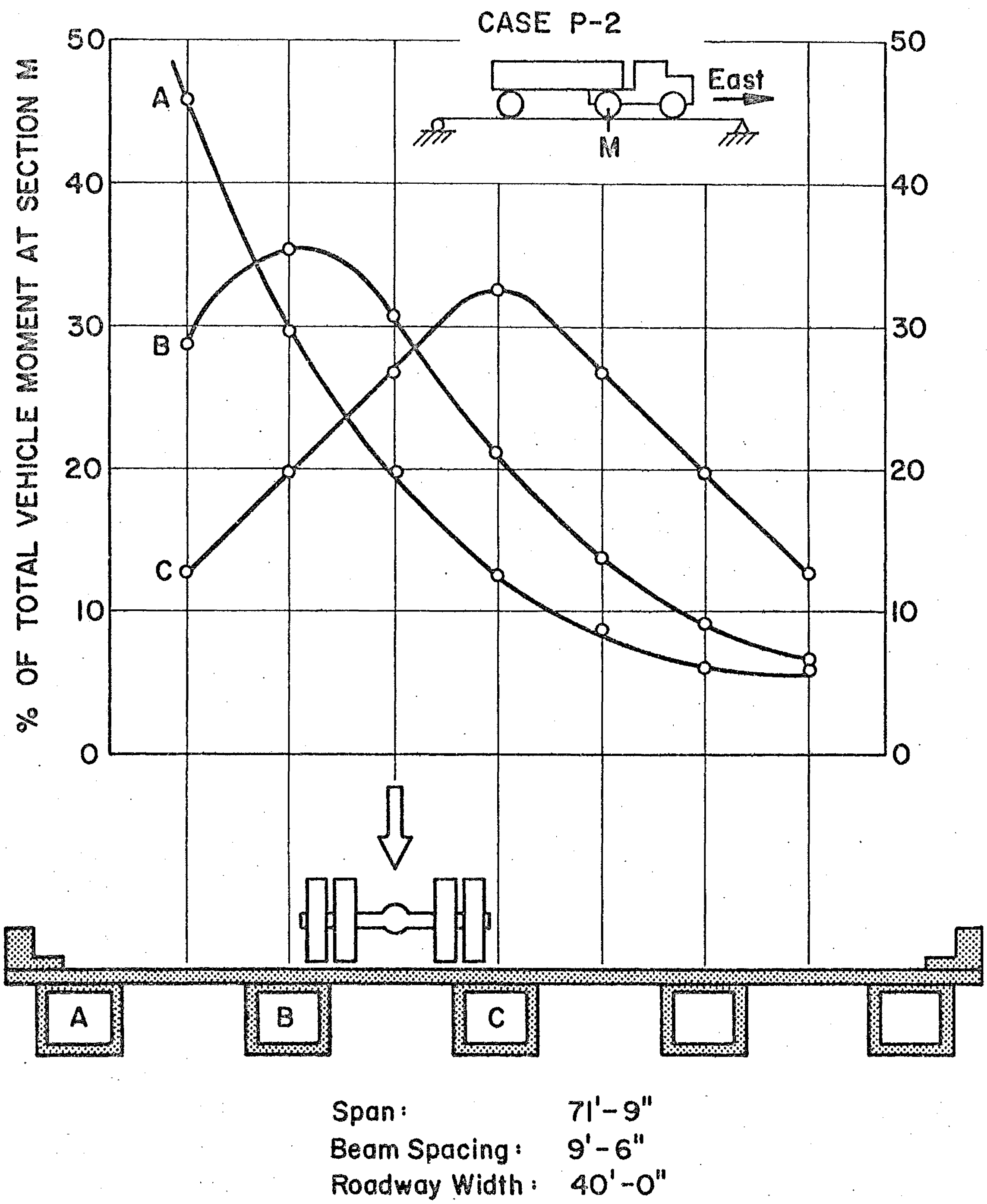
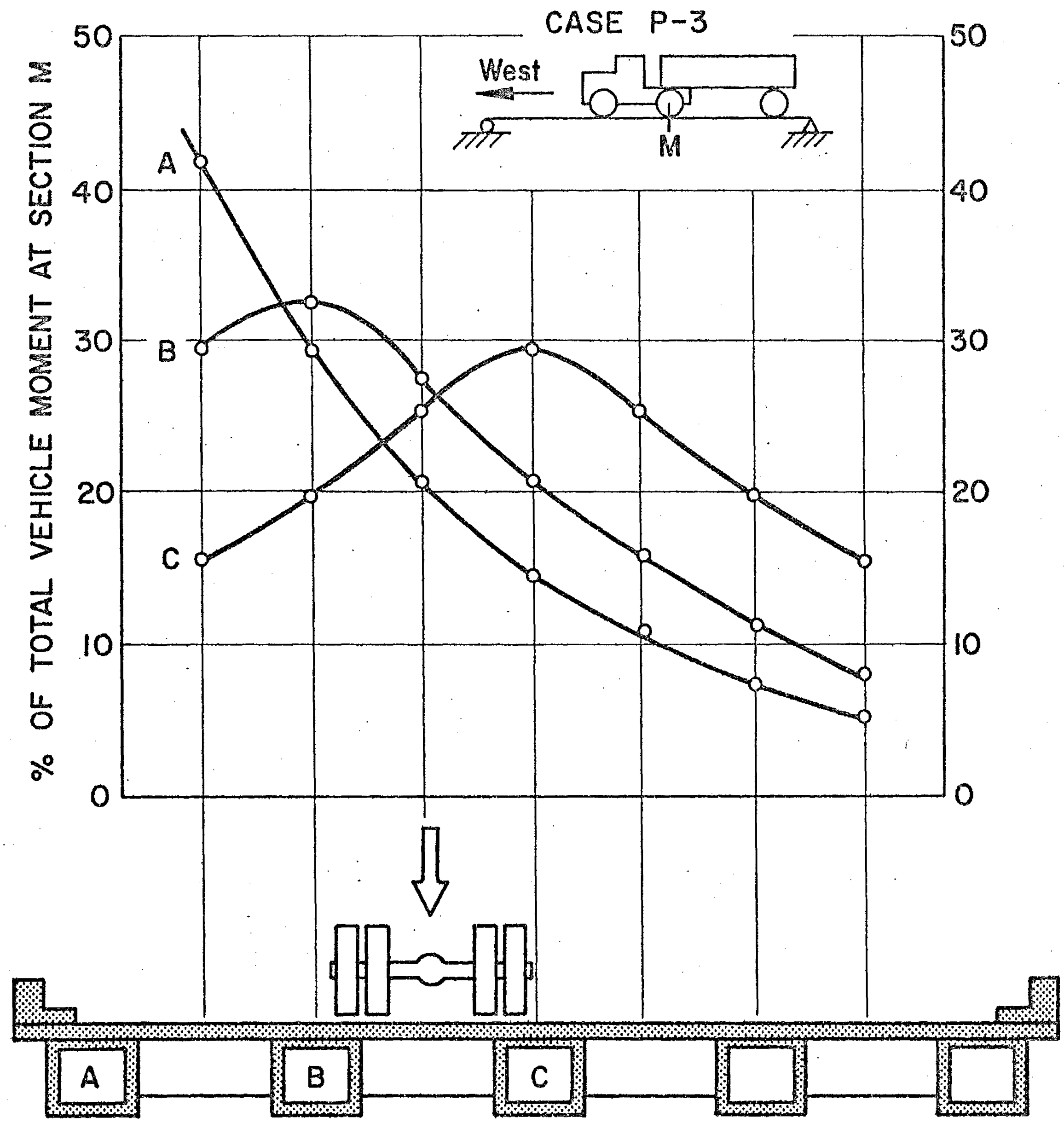


Fig. 9 Influence Lines for Bending Moments in Beams
Philadelphia Bridge (Midspan diaphragms removed)
Section M - Case P-2



Span: 71'-9"
Beam Spacing: 9'-6"
Roadway Width: 40'-0"

Fig. 10 Influence Lines for Bending Moments in Beams Philadelphia Bridge (Midspan diaphragms in place) Section M - Case P-3

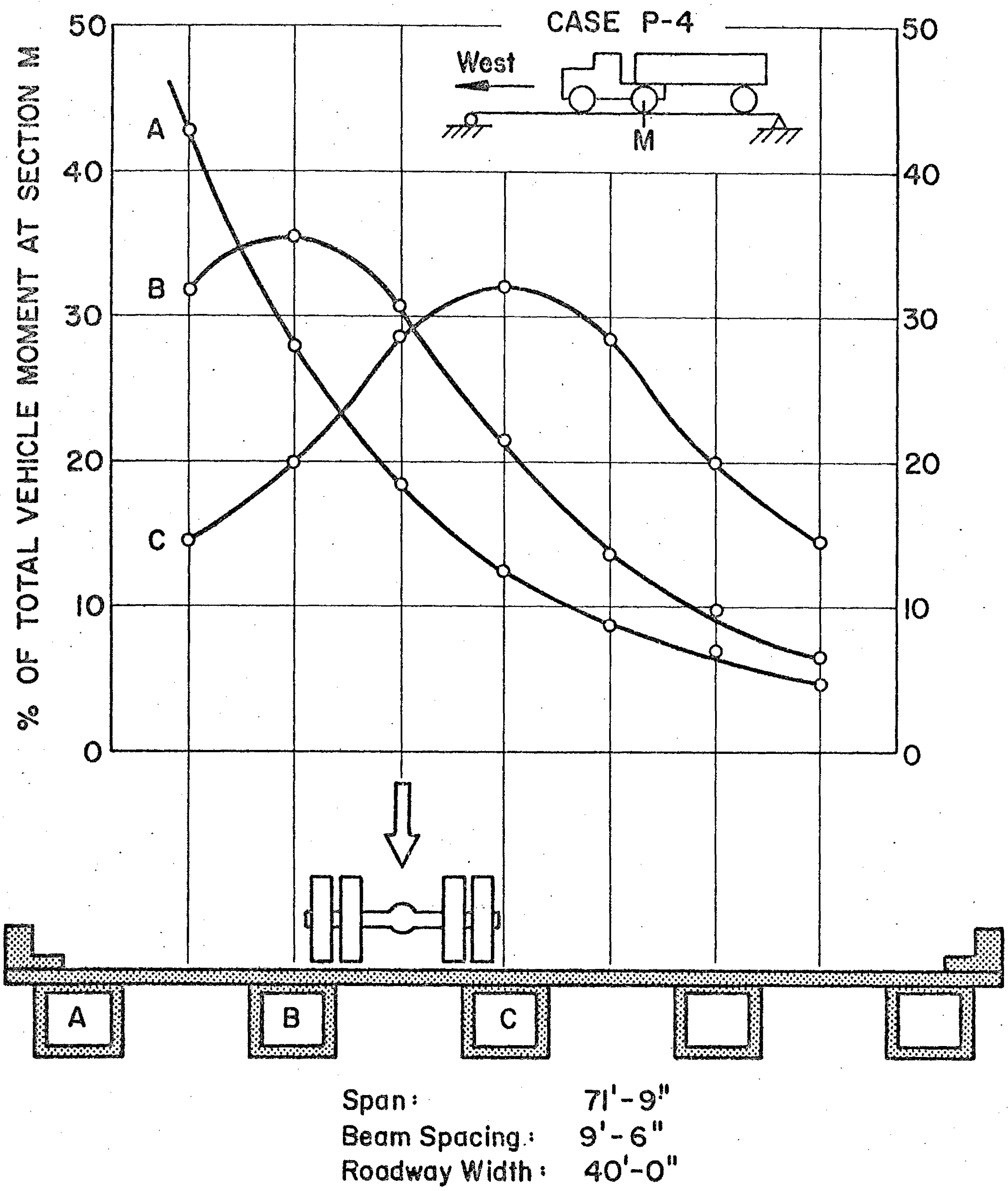
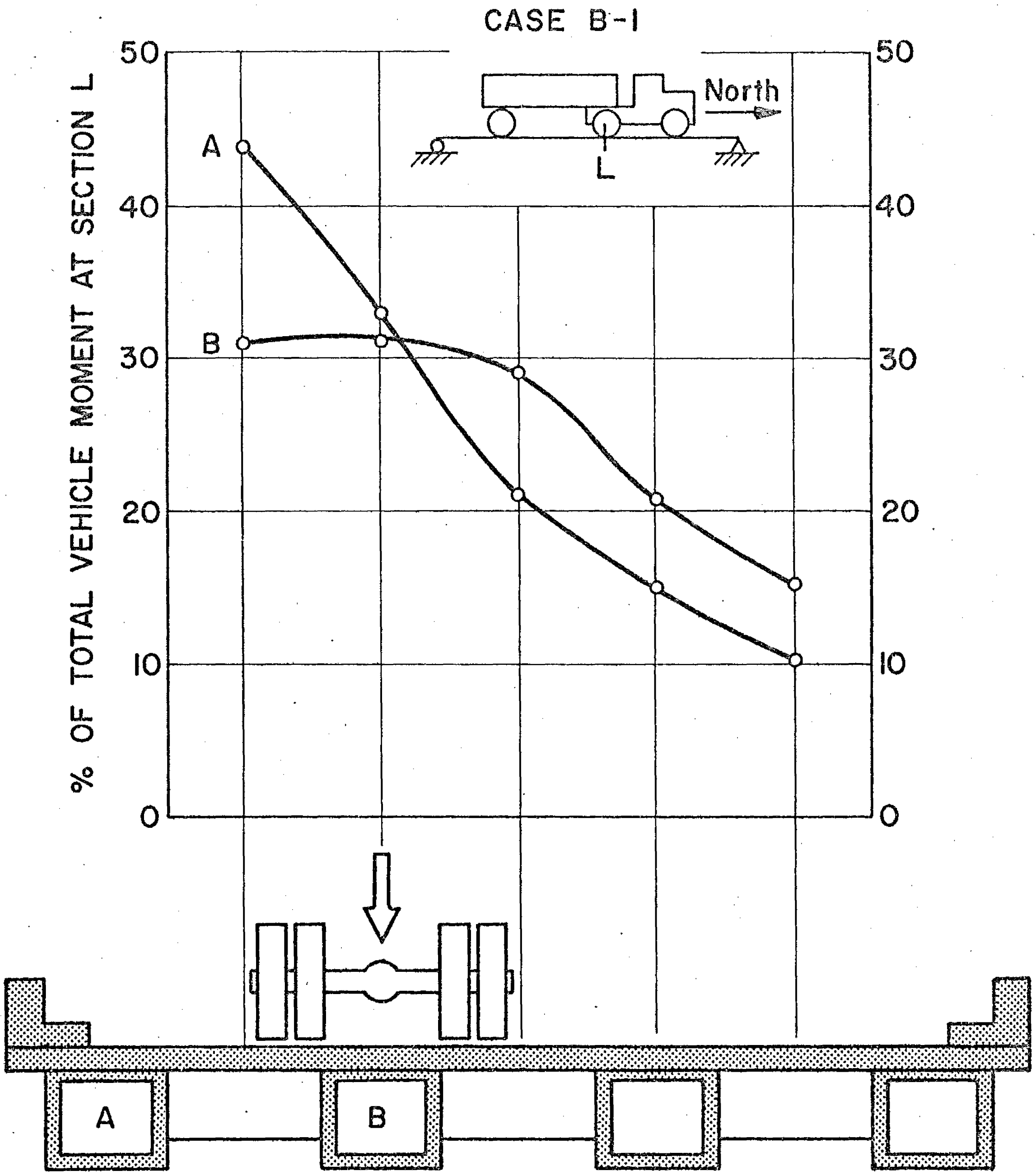


Fig. 11 Influence Lines for Bending Moments in Beams
Philadelphia Bridge (Midspan diaphragms removed)
Section M - Case P-4



Span: 65'-3"
Beam Spacing: 8'-9 3/8"
Roadway Width: 28'-0"

Fig. 12 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section L - Case B-1

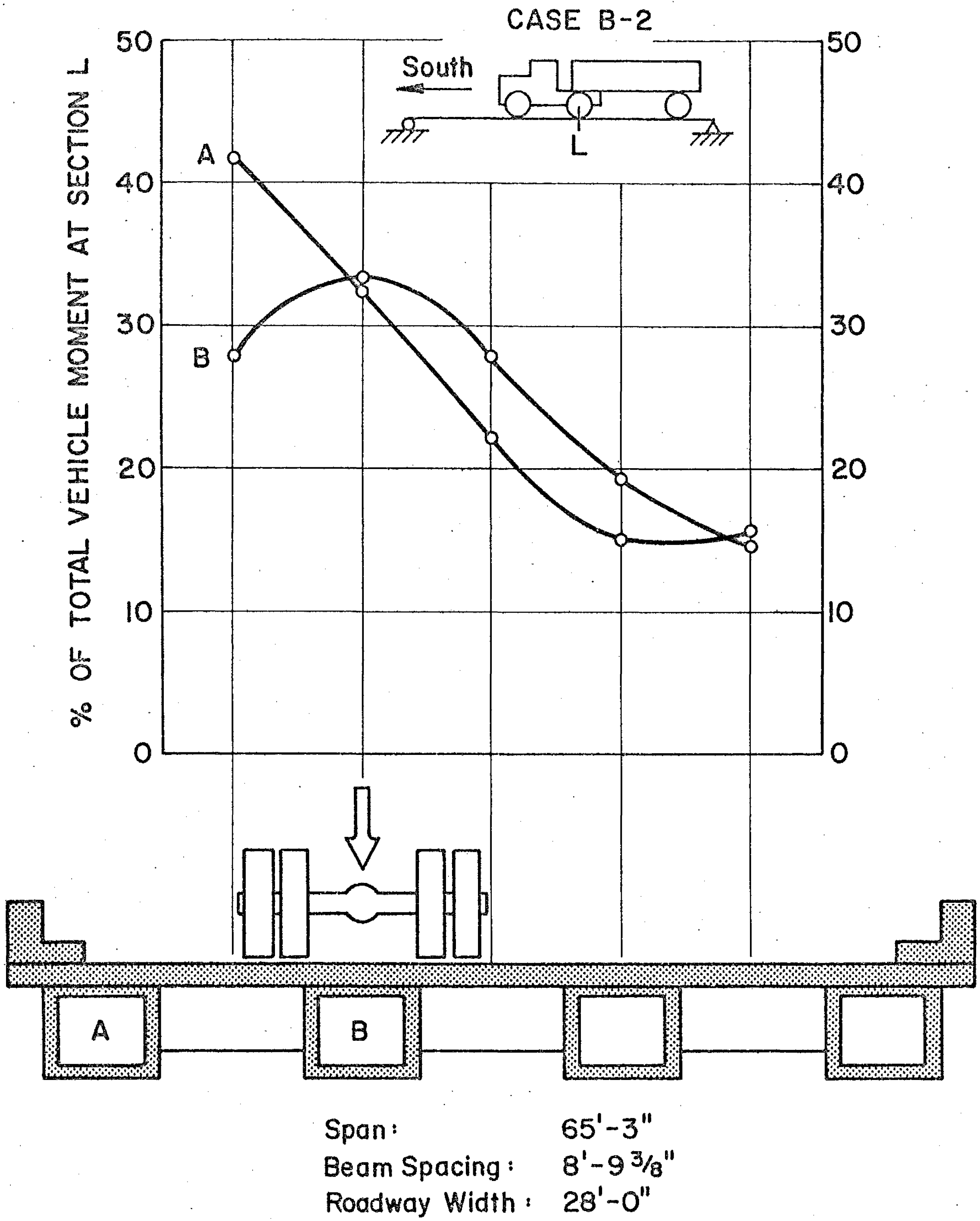


Fig. 13 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section L - Case B-2

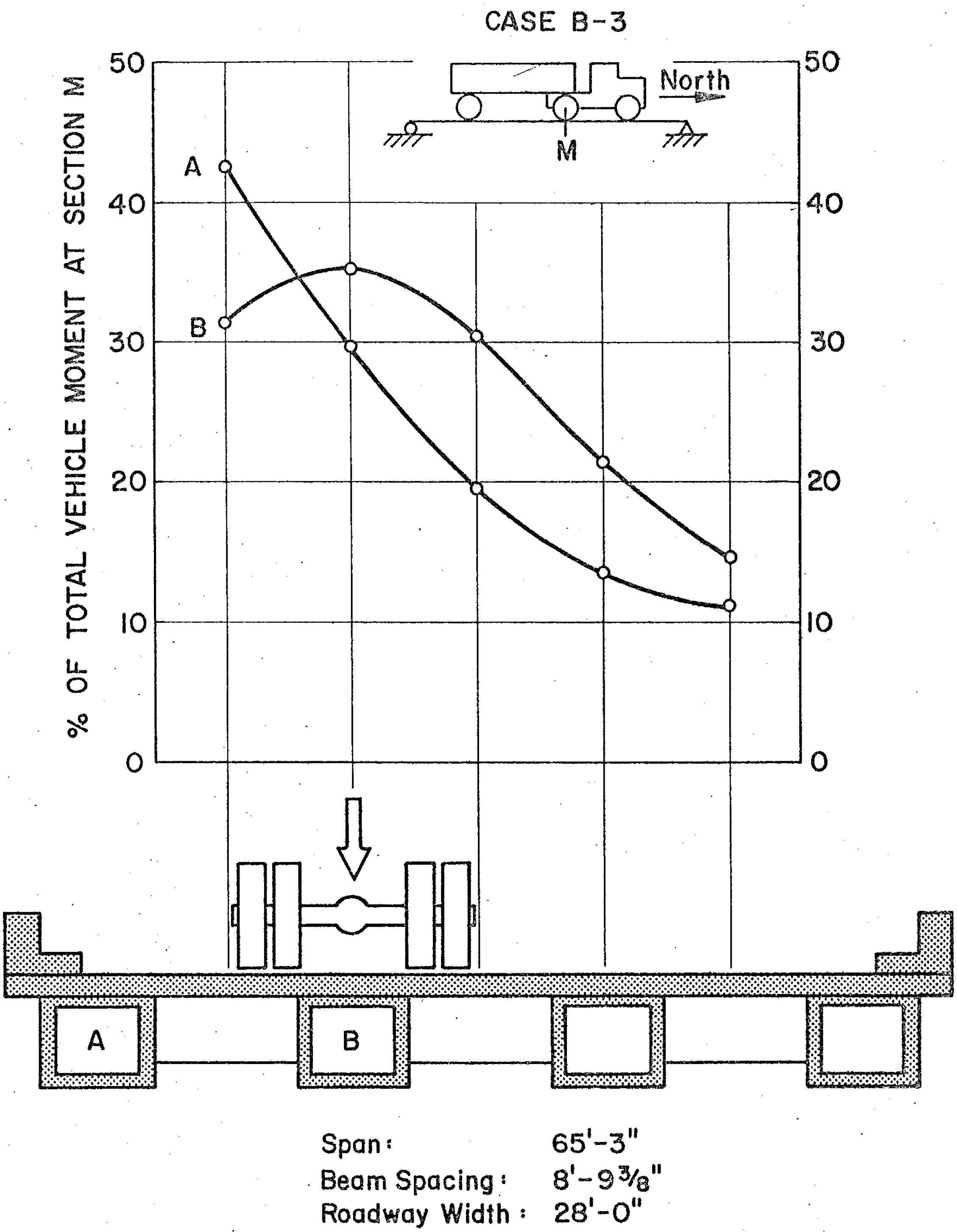
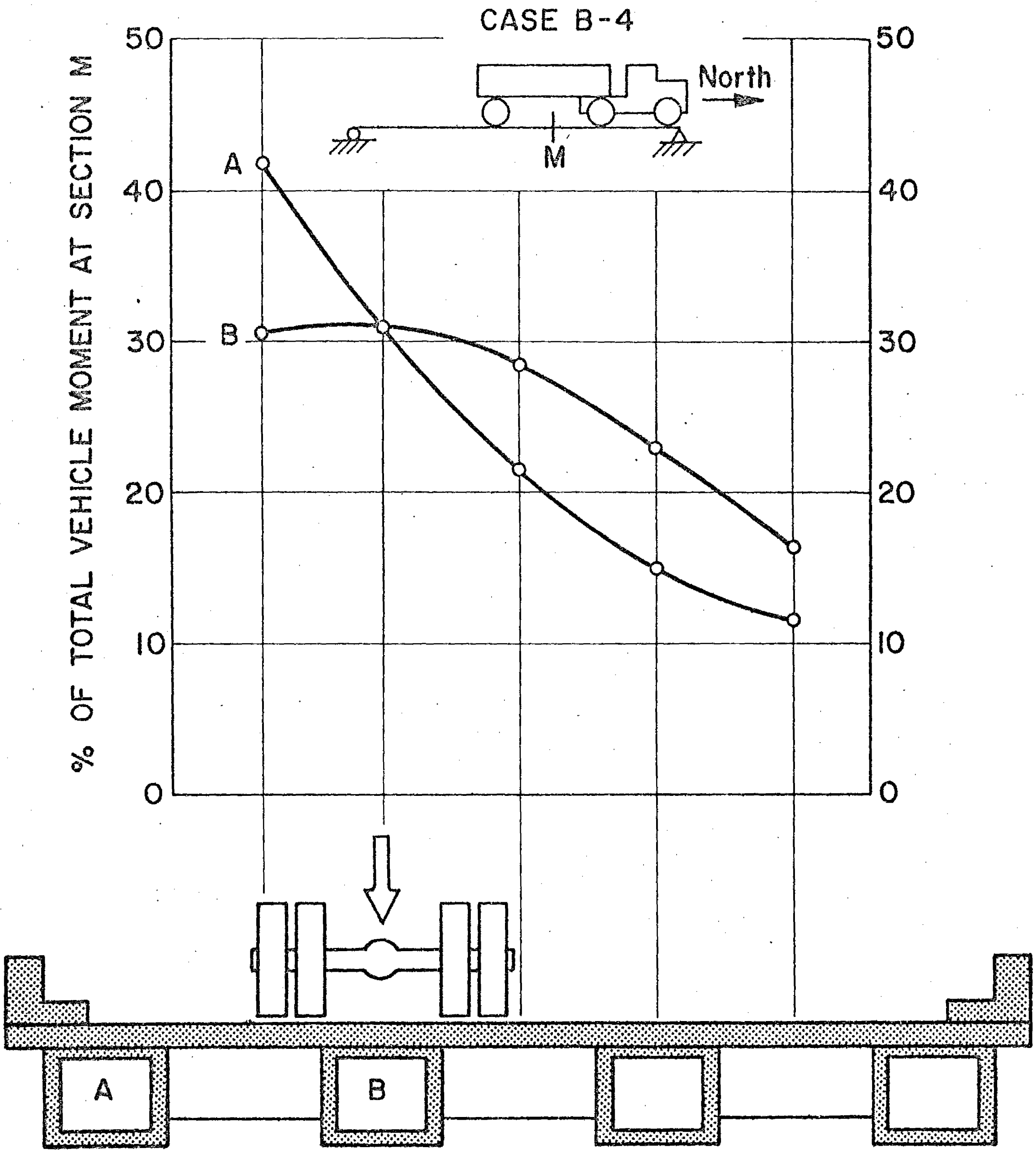
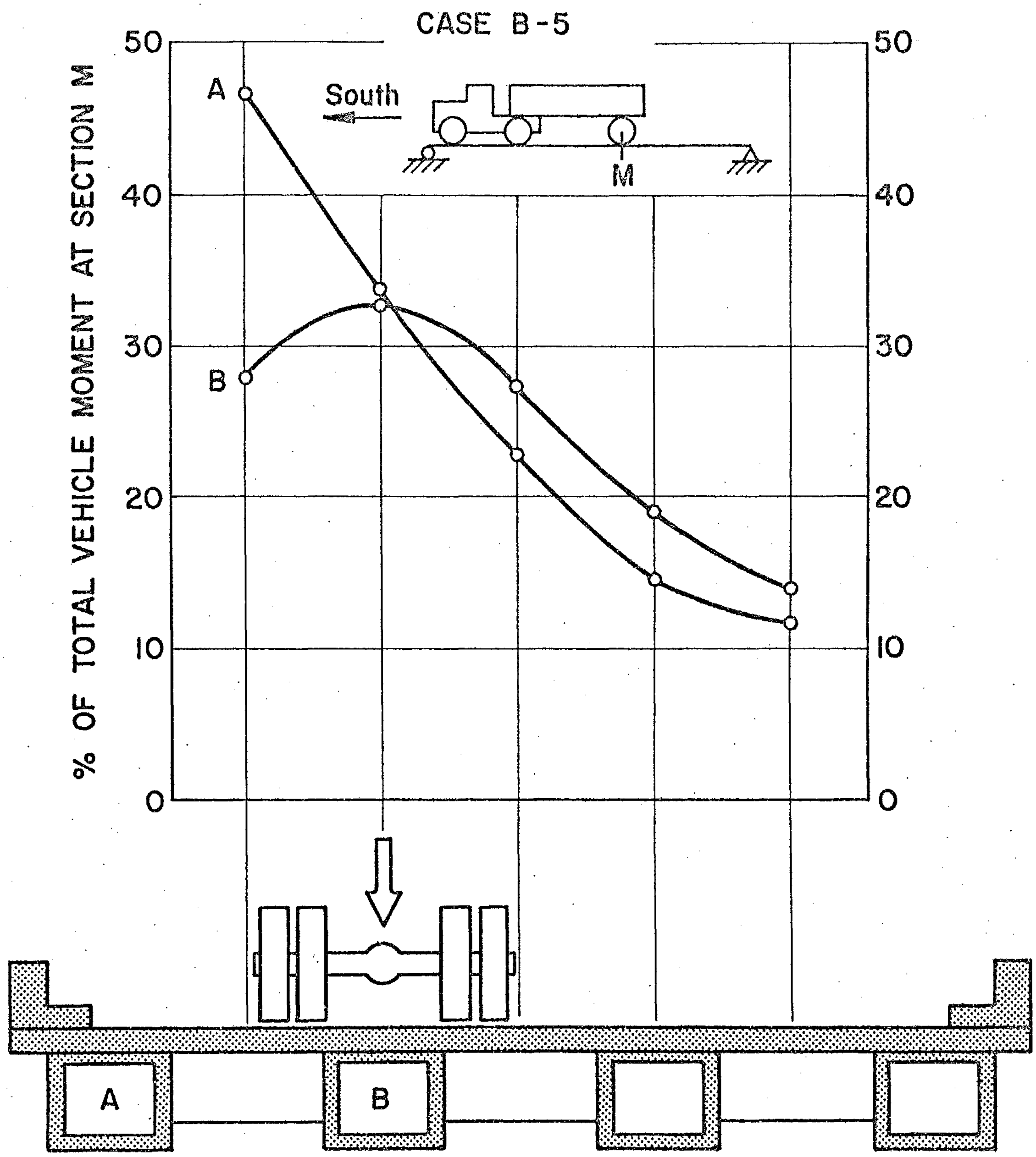


Fig. 14 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section M - Case B-3



Span: 65'-3"
Beam Spacing: 8'-9⅜"
Roadway Width: 28'-0"

Fig. 15 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section M - Case B-4



Span: 65'-3

Beam Spacing: 8'-9⅜"

Roadway Width: 28'-0"

Fig. 16 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section M - Case B-5

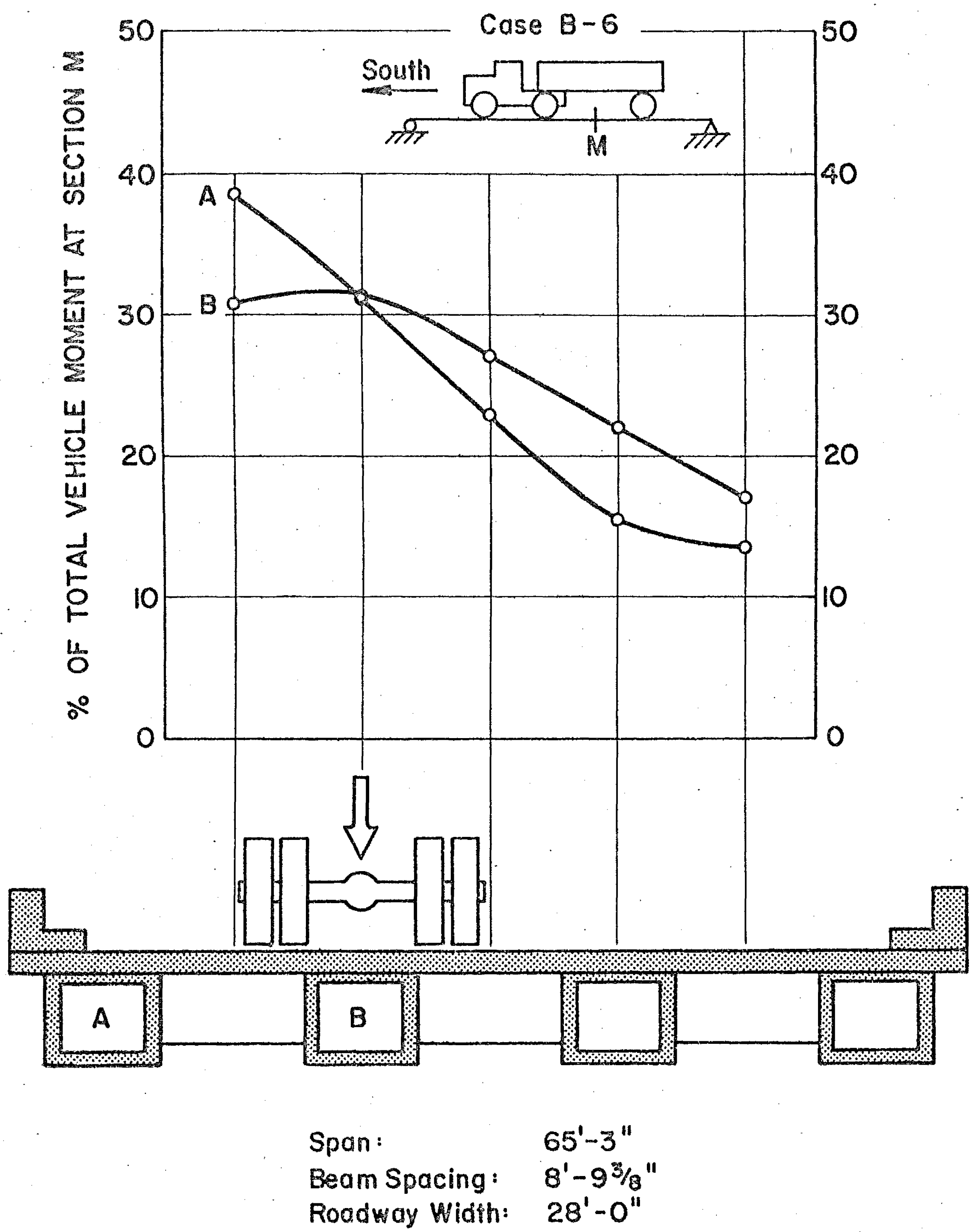


Fig. 17 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section M - Case B-6

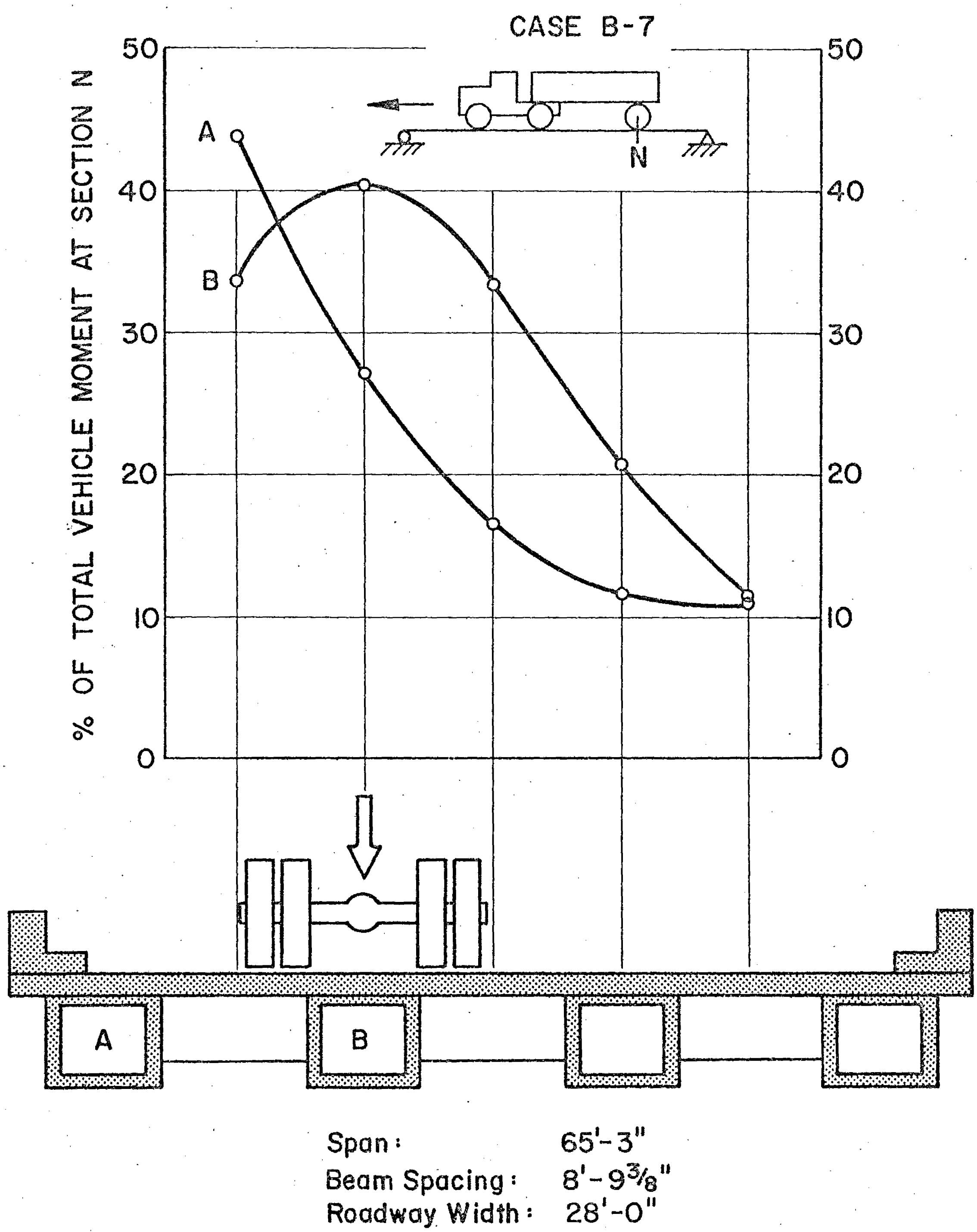


Fig. 18 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section N - Case B-7

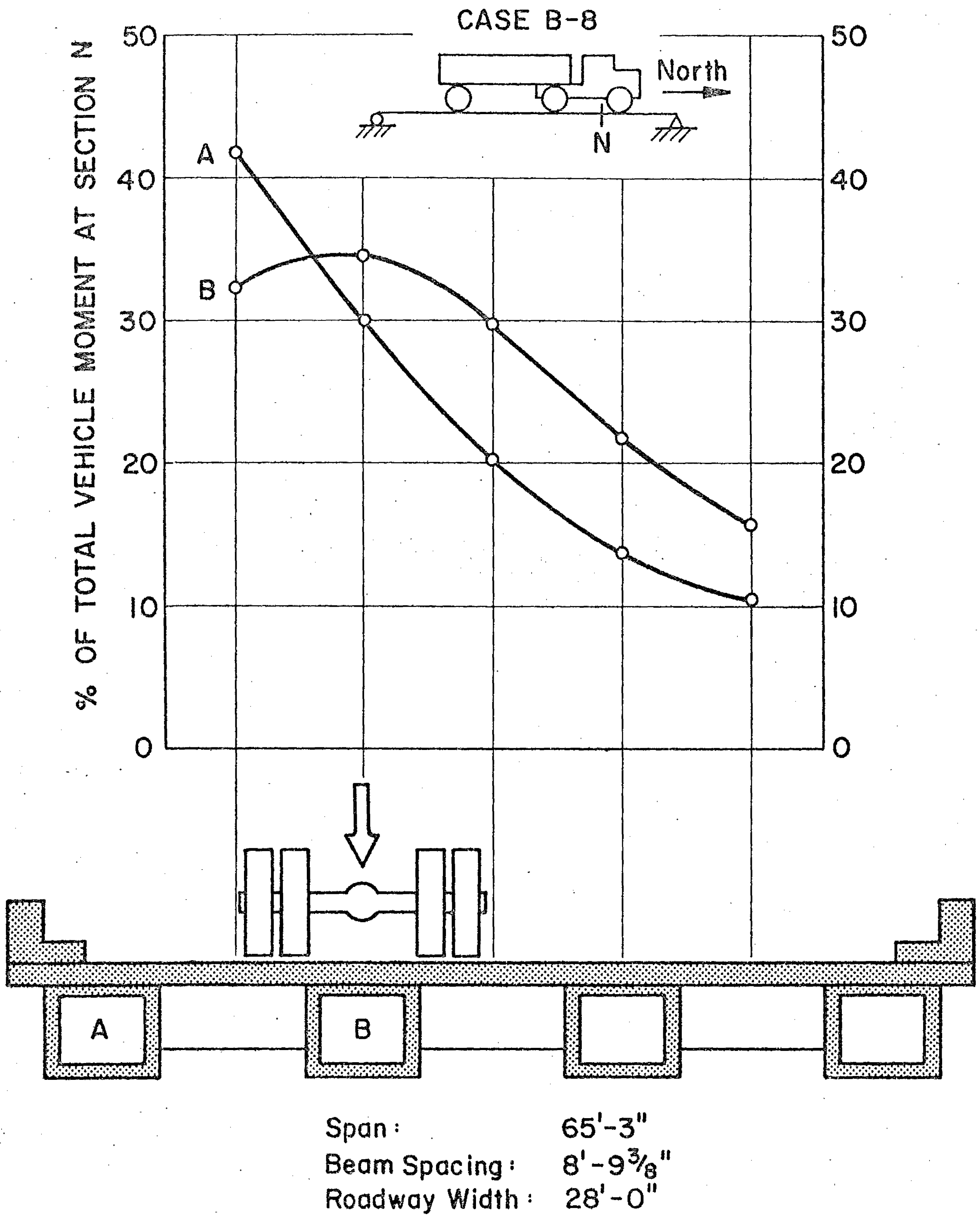


Fig. 19 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section N - Case B-8

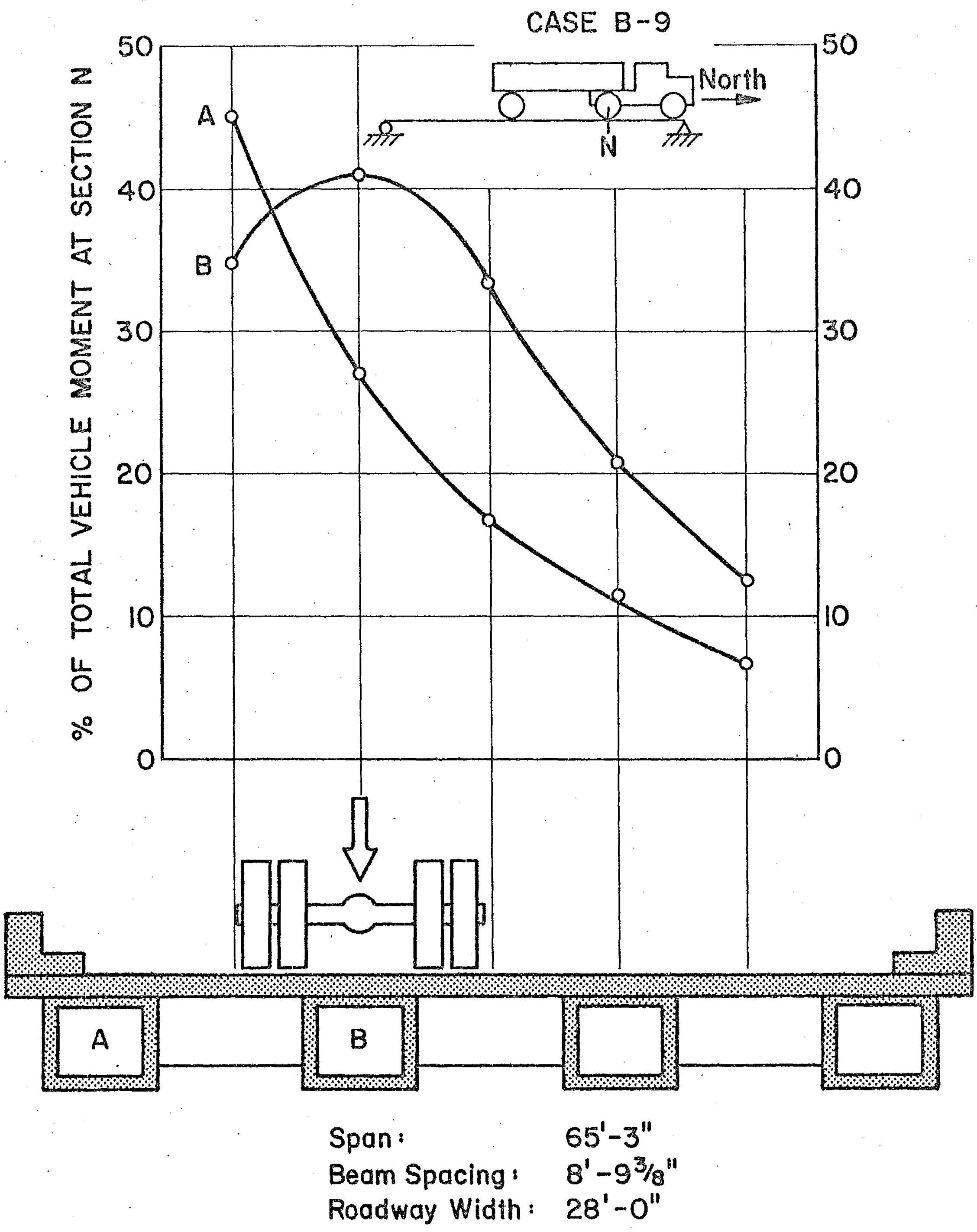


Fig. 20 Influence Lines for Bending Moments in Beams
Berwick Bridge - Section N - Case B-9

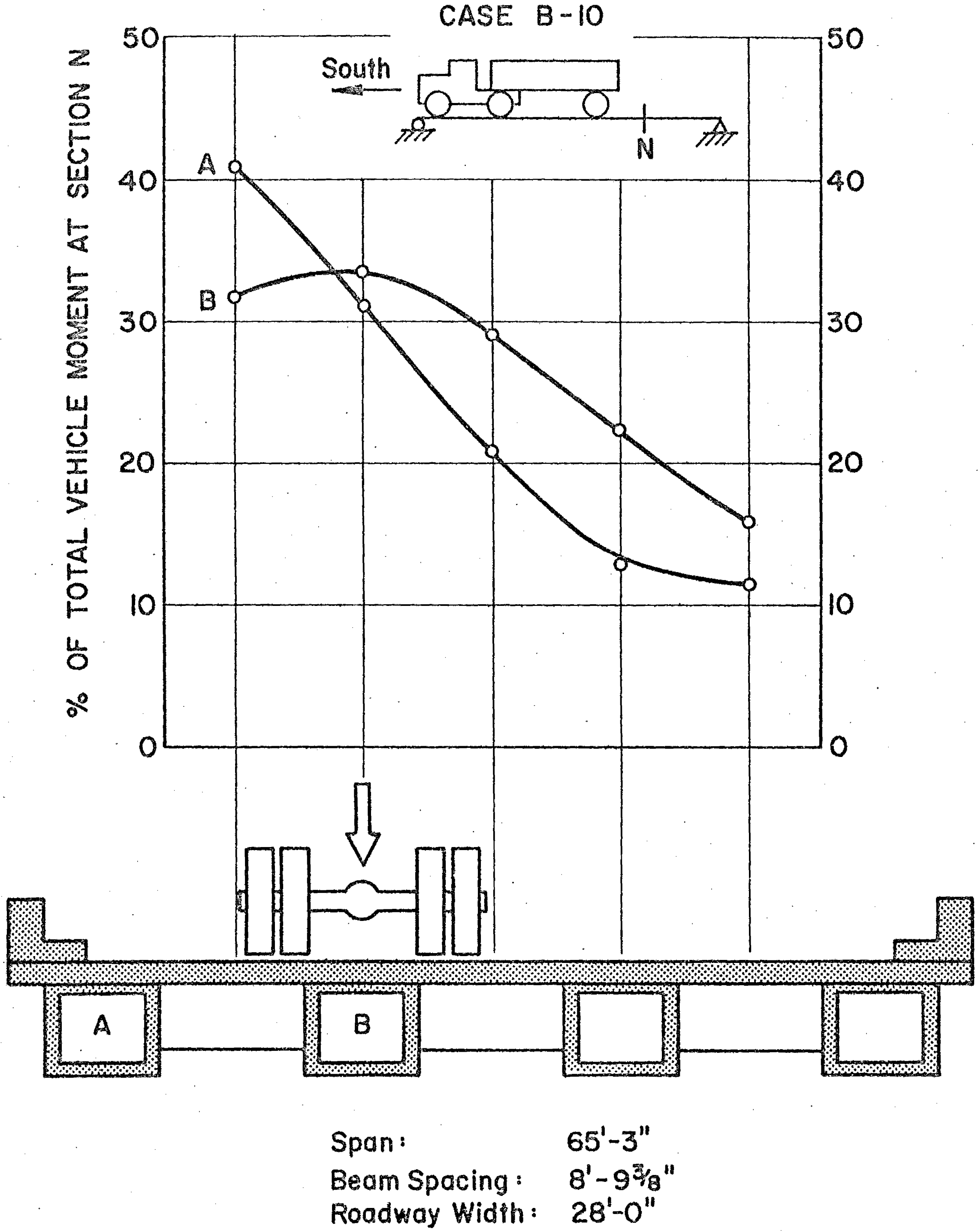
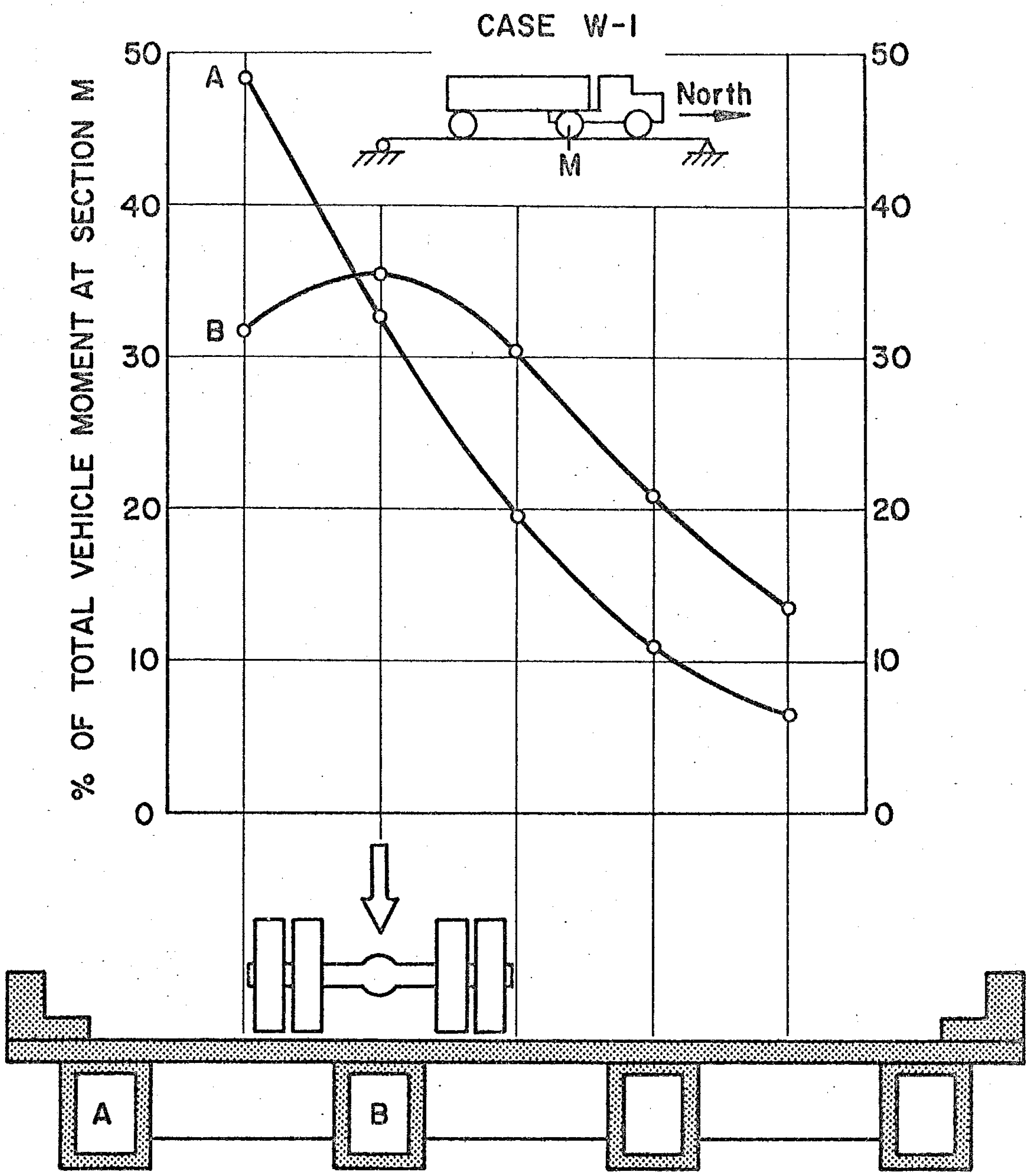
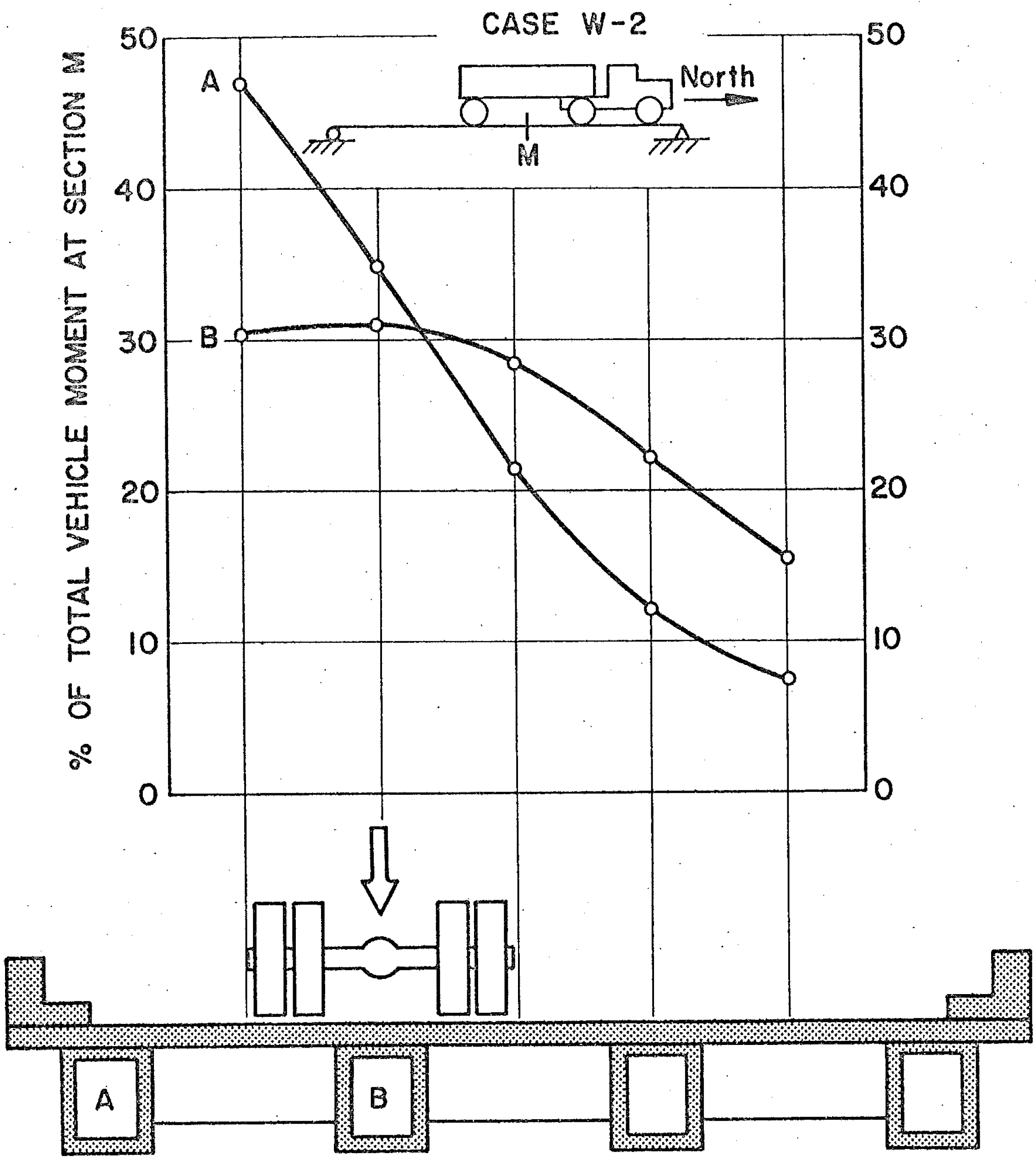


Fig. 21 Influence Lines for Bending Moments in Beams Berwick Bridge - Section N - Case B-10



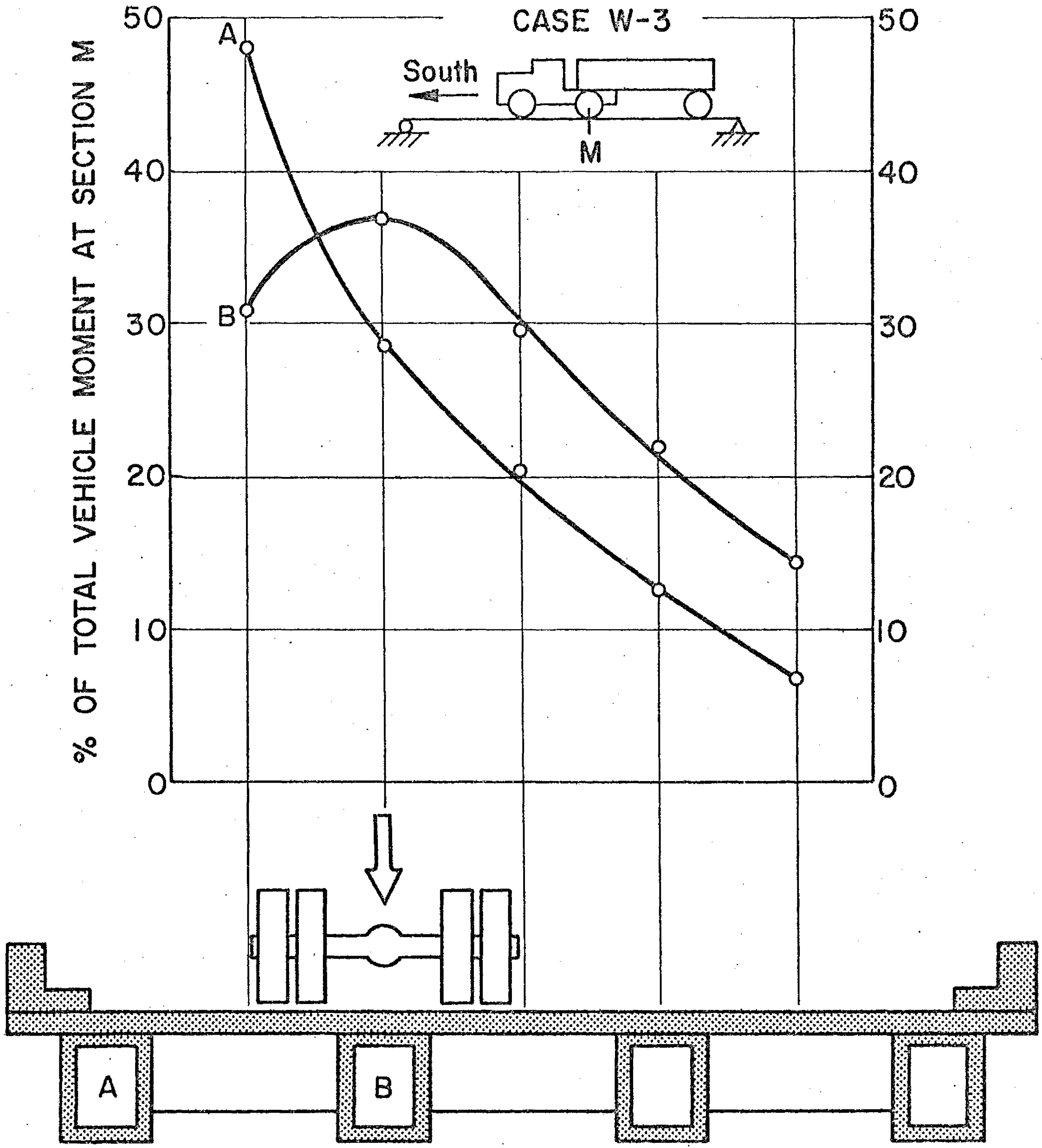
Span: 64'-8"
Beam Spacing: 9'-0"
Roadway Width 28'-0"

Fig. 22 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section M - Case W-1



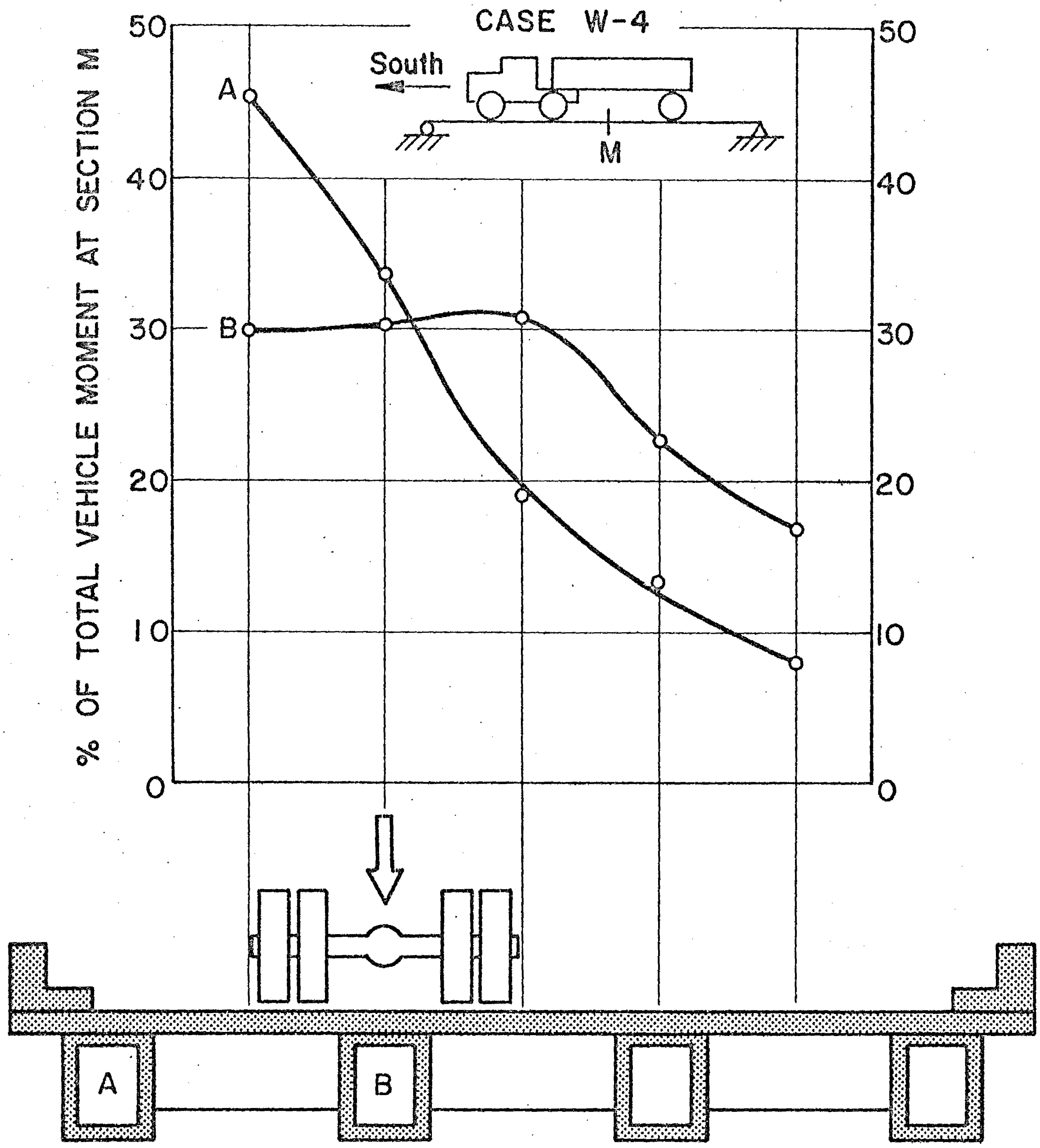
Span:	64'-8"
Beam Spacing:	9'-0"
Roadway Width:	28'-0"

Fig. 23 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section M - Case W-2



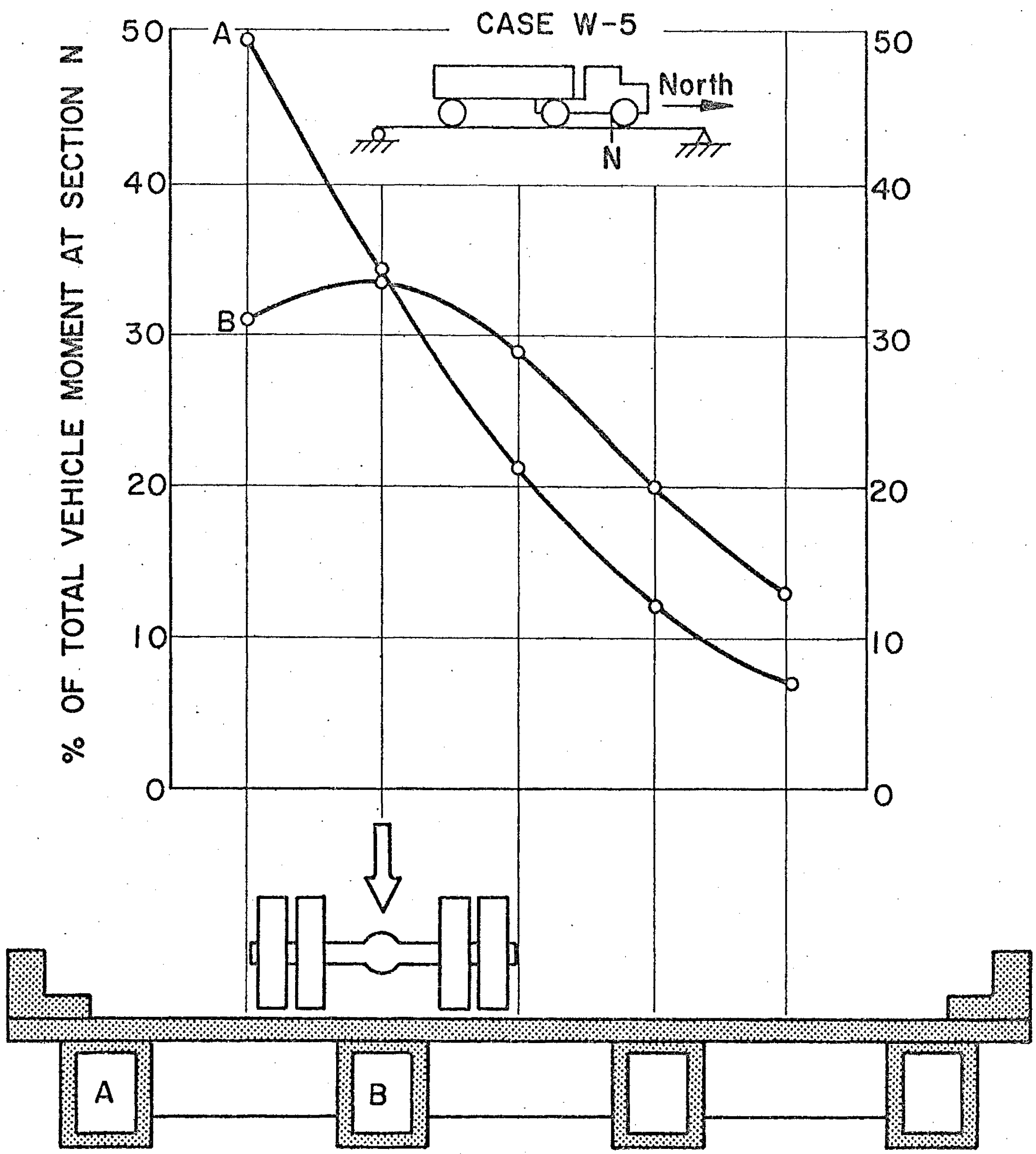
Span: 64'-8"
Beam Spacing: 9'-0"
Roadway Width: 28'-0"

Fig. 24 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section M - Case W-3



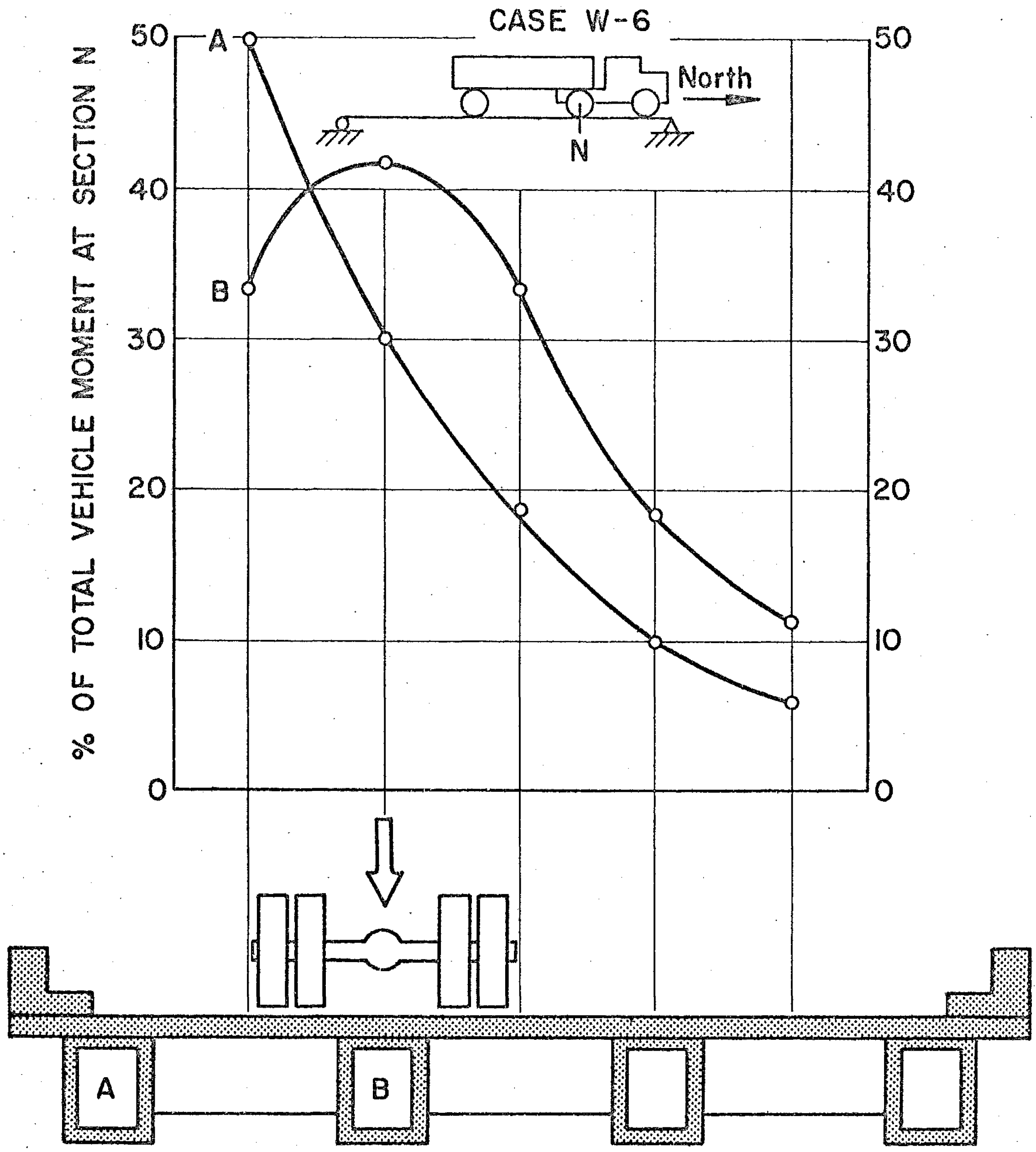
Span: 64'-8"
Beam Spacing: 9'-0"
Roadway Width: 28'-0"

Fig. 25 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section M - Case W-4



Span: 64'-8"
Beam Spacing: 9'-0"
Roadway Width: 28'-0"

Fig. 26 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section N - Case W-5



Span: 64'-8"
Beam Spacing 9'-0"
Roadway Width: 28'-0"

Fig. 27 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section N - Case W-6

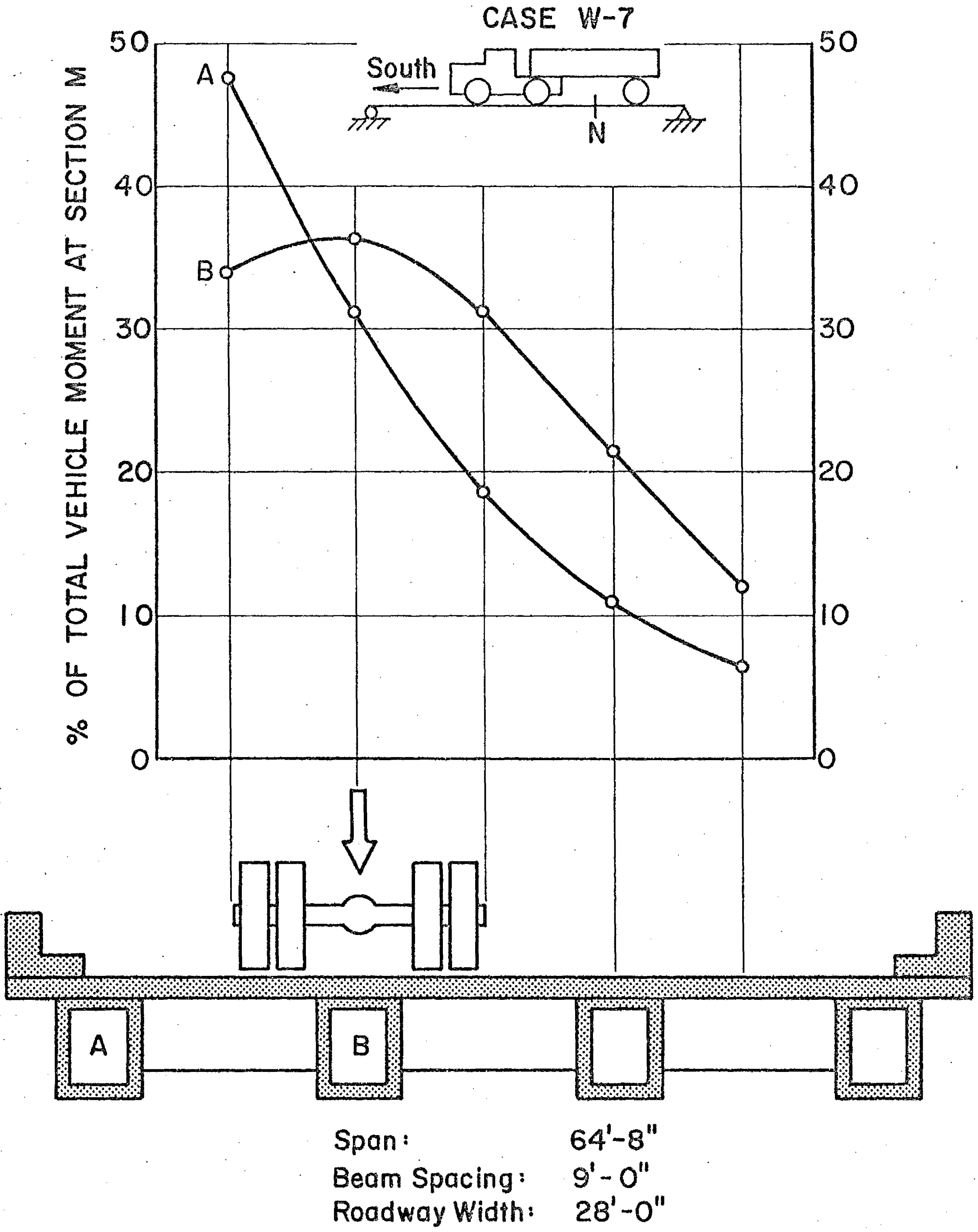
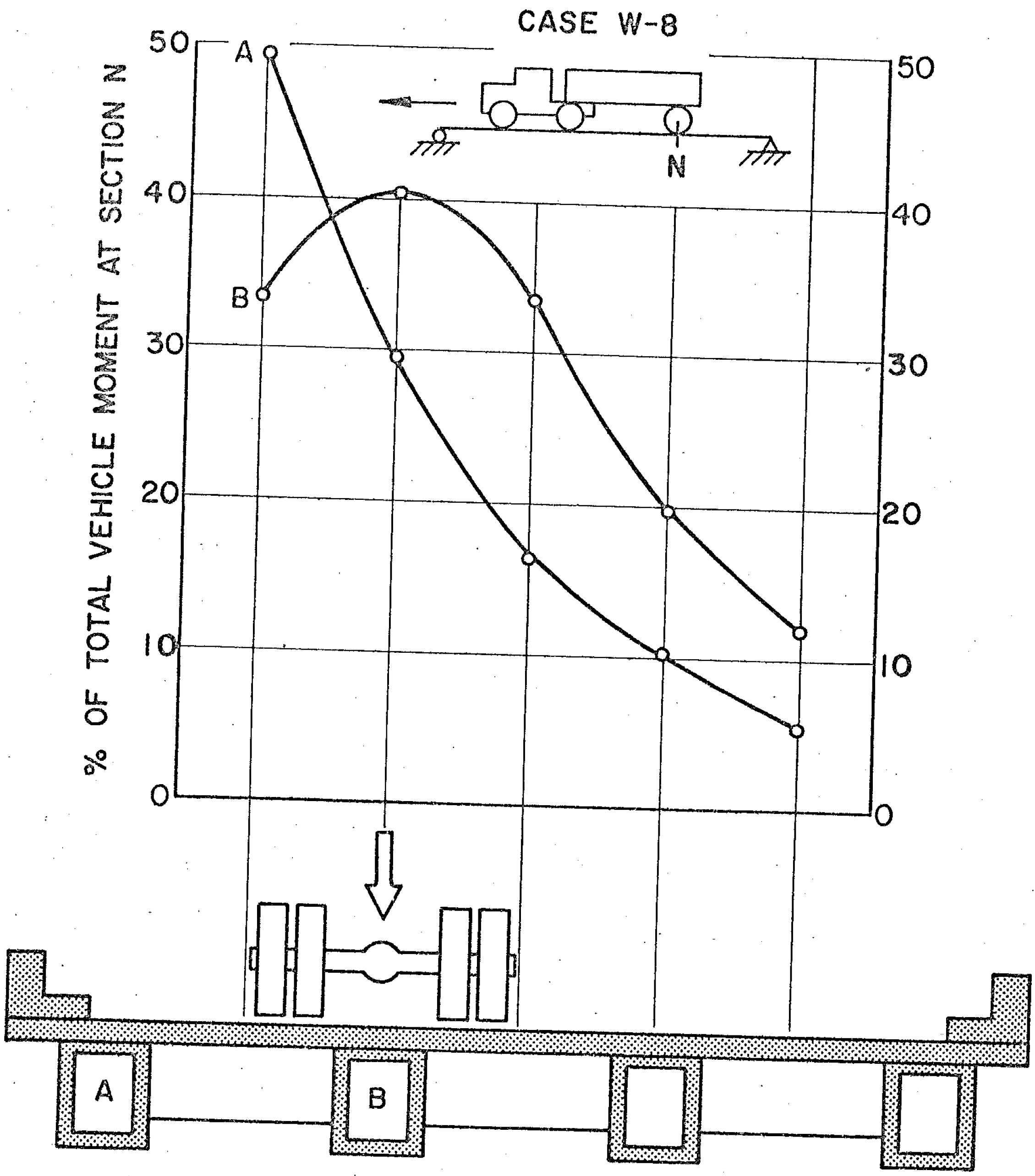


Fig. 28 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section N - Case W-7



Span: 64'-8"
Beam Spacing: 9'-0"
Roadway Width: 28'-0"

Fig. 29 Influence Lines for Bending Moments in Beams
White Haven Bridge - Section N - Case W-8

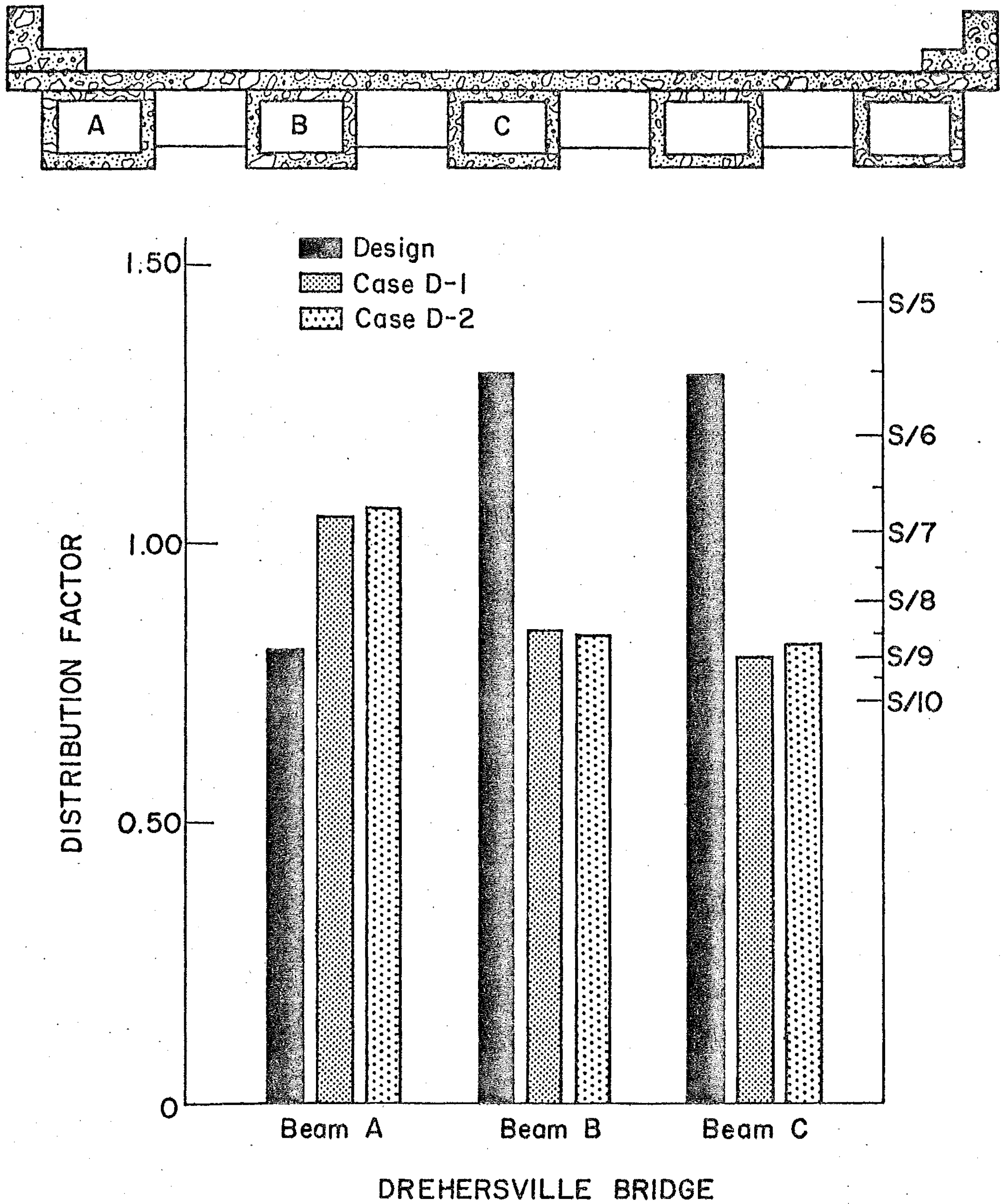


Fig. 30 Experimentally Developed Distribution Factors and Design Values: Drehersville Bridge - Section M

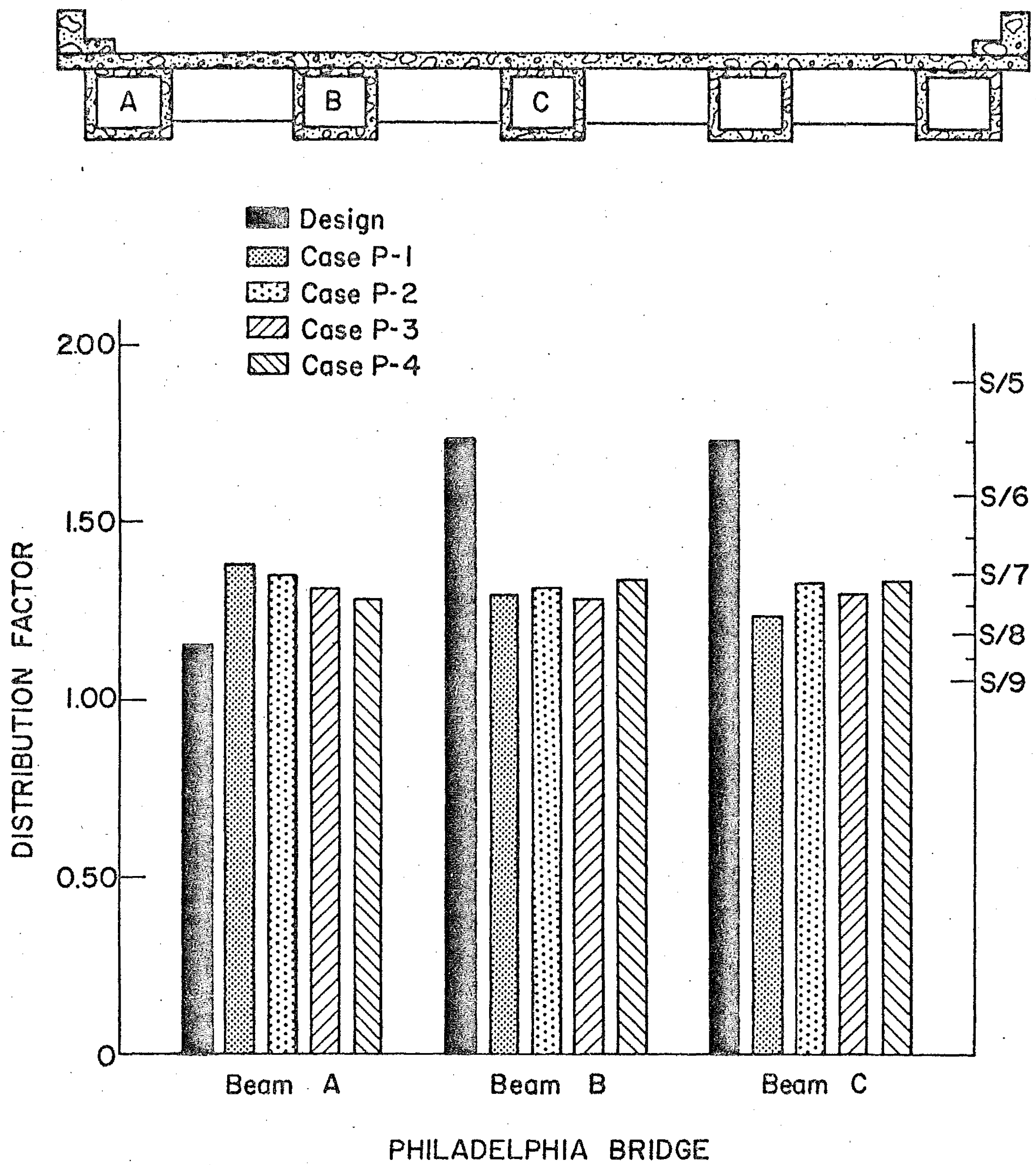


Fig. 31 Experimentally Developed Distribution Factors and Design Values: Philadelphia Bridge - Section M

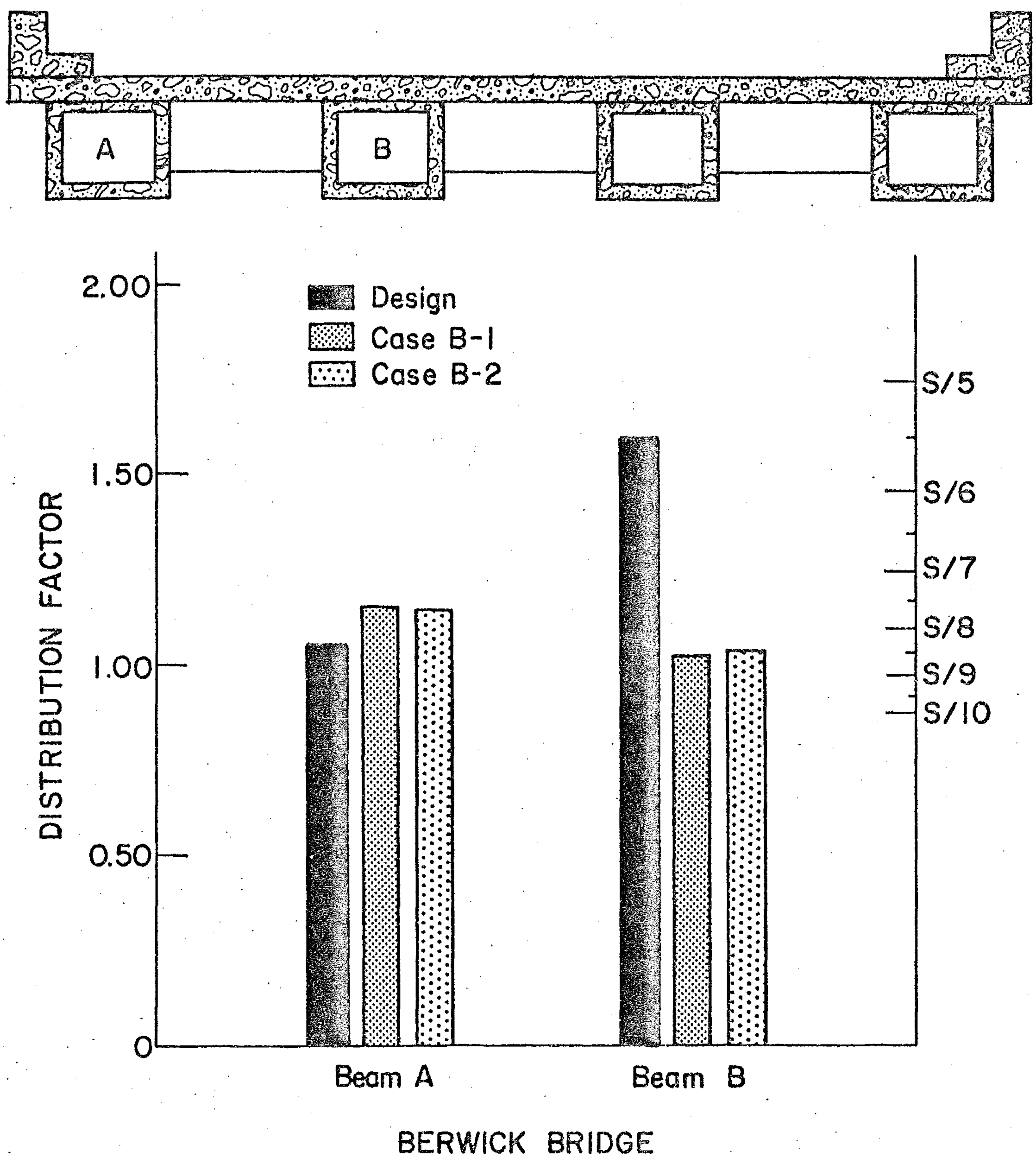


Fig. 32 Experimentally Developed Distribution Factors and Design Values: Berwick Bridge - Section L

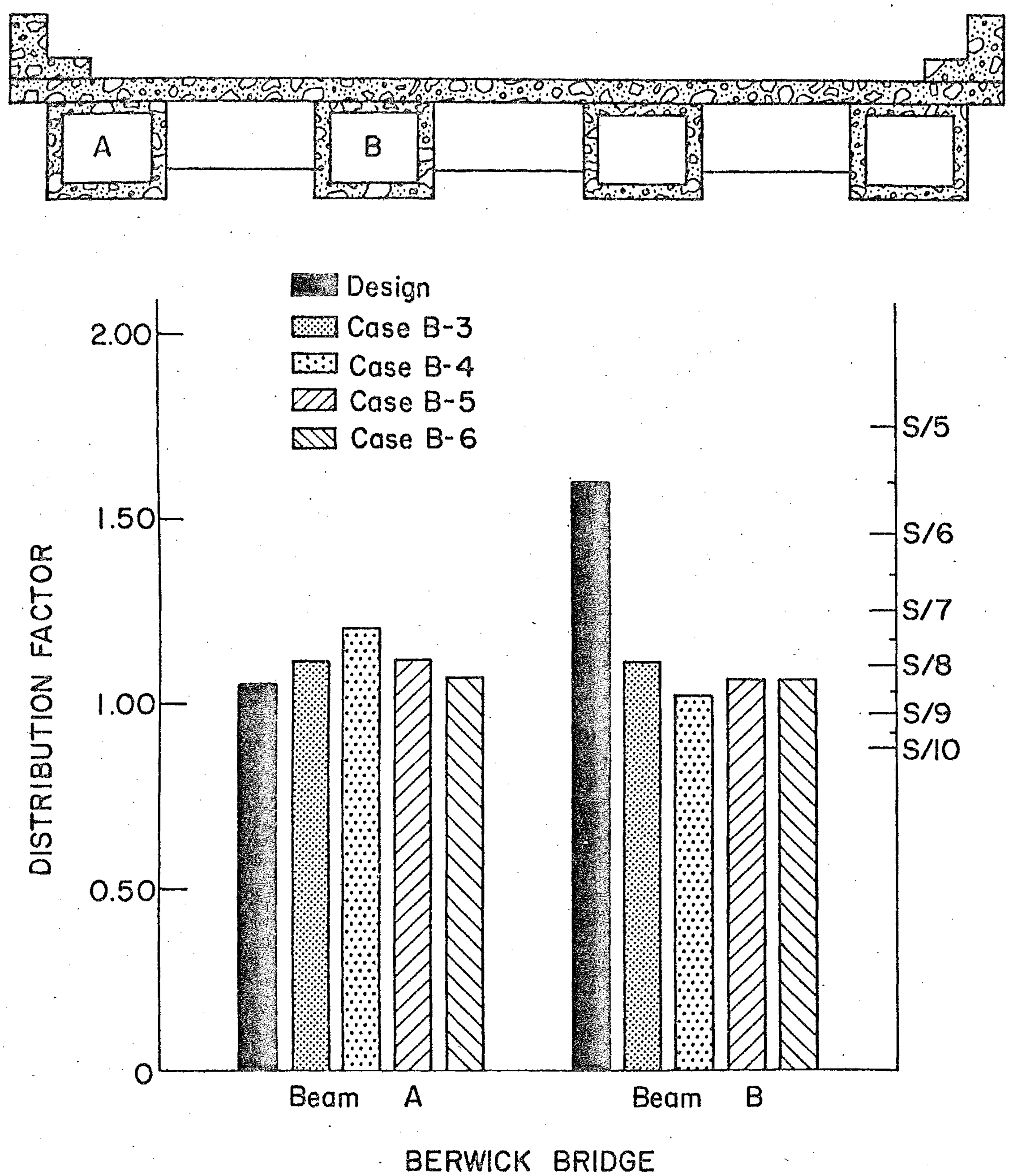


Fig. 33 Experimentally Developed Distribution Factors and Design Values: Berwick Bridge - Section M

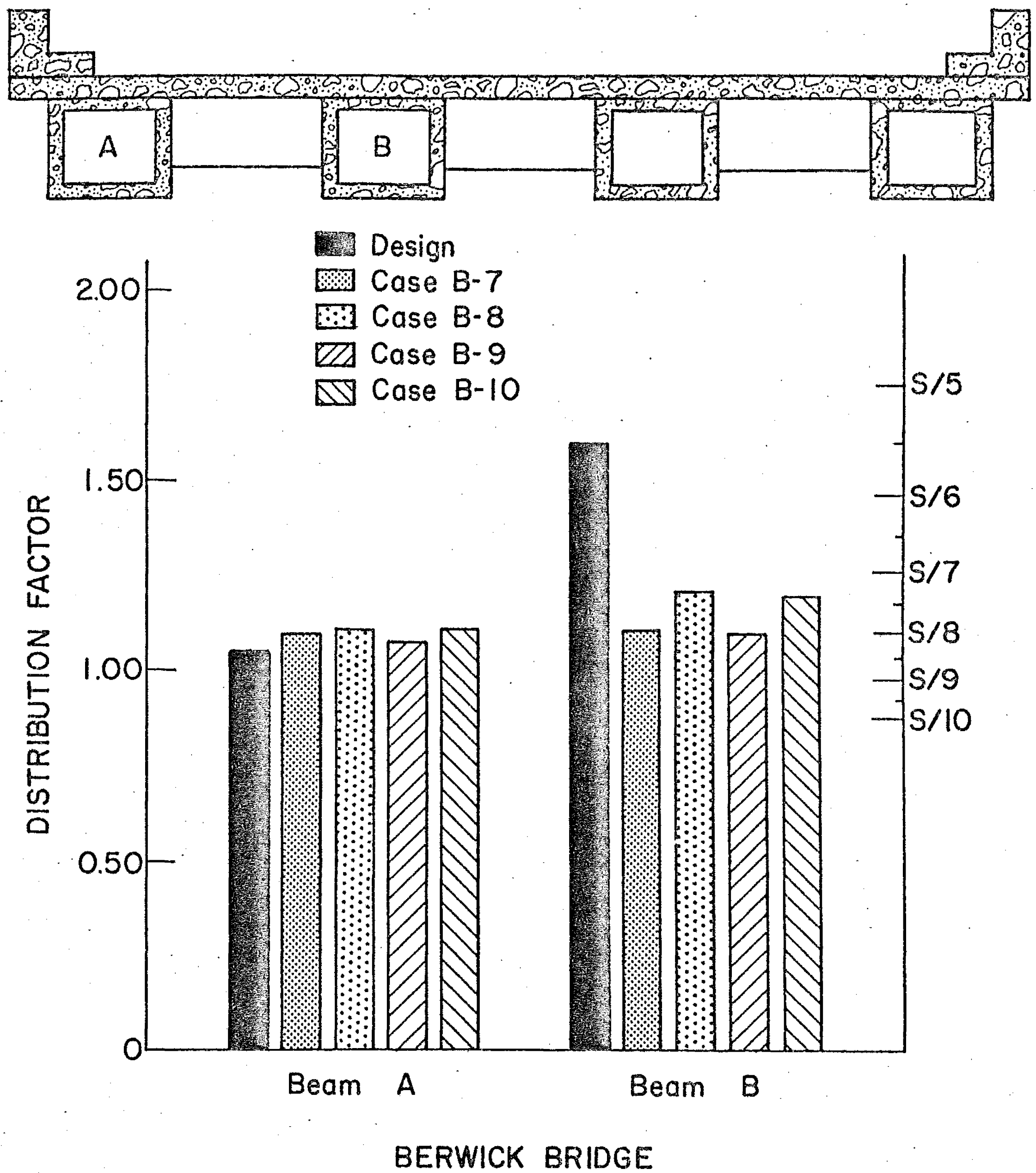


Fig. 34 Experimentally Developed Distribution Factors and Design Values: Berwick Bridge - Section N

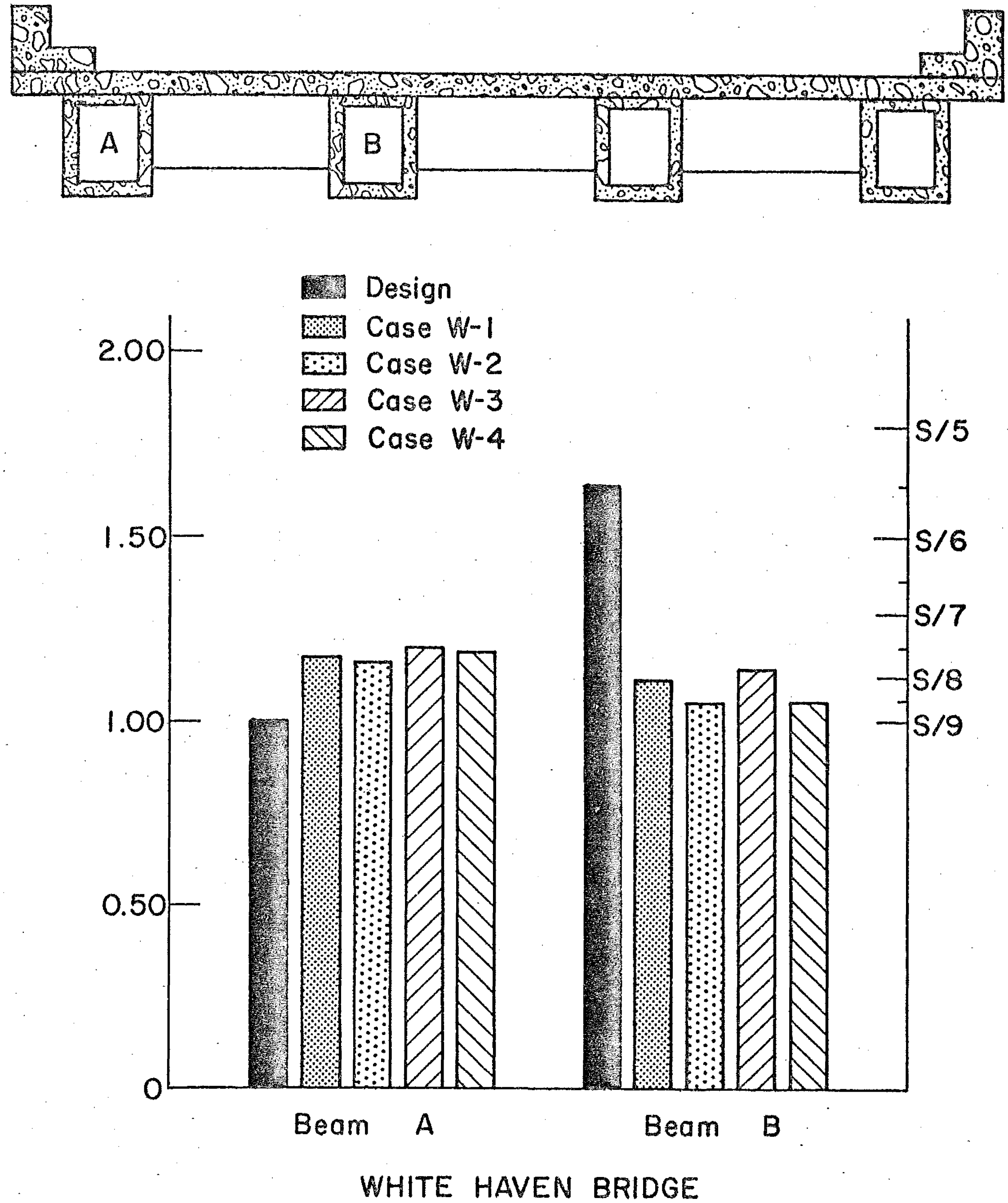


Fig. 35 Experimentally Developed Distribution Factors and Design Values: White Haven Bridge - Section M

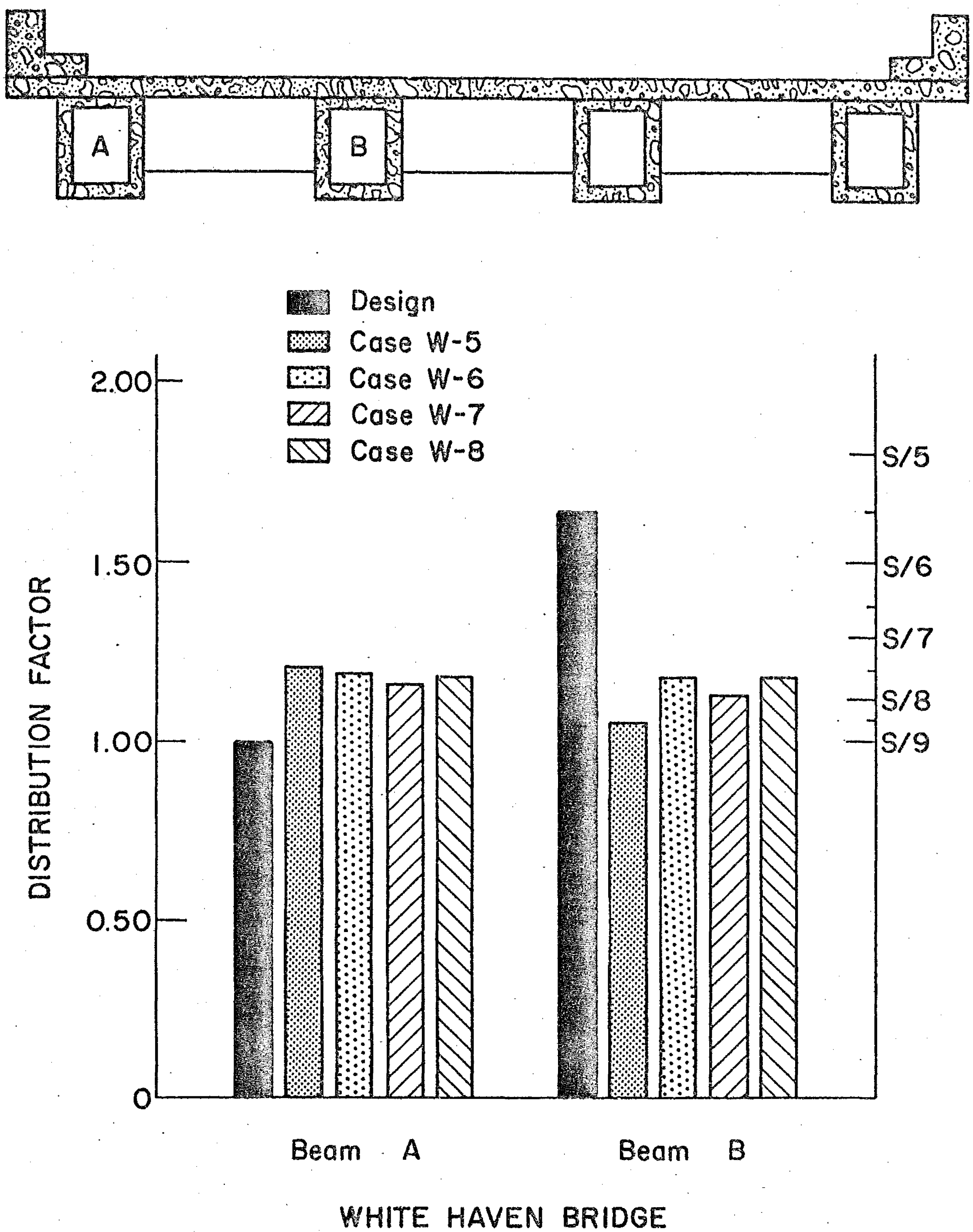
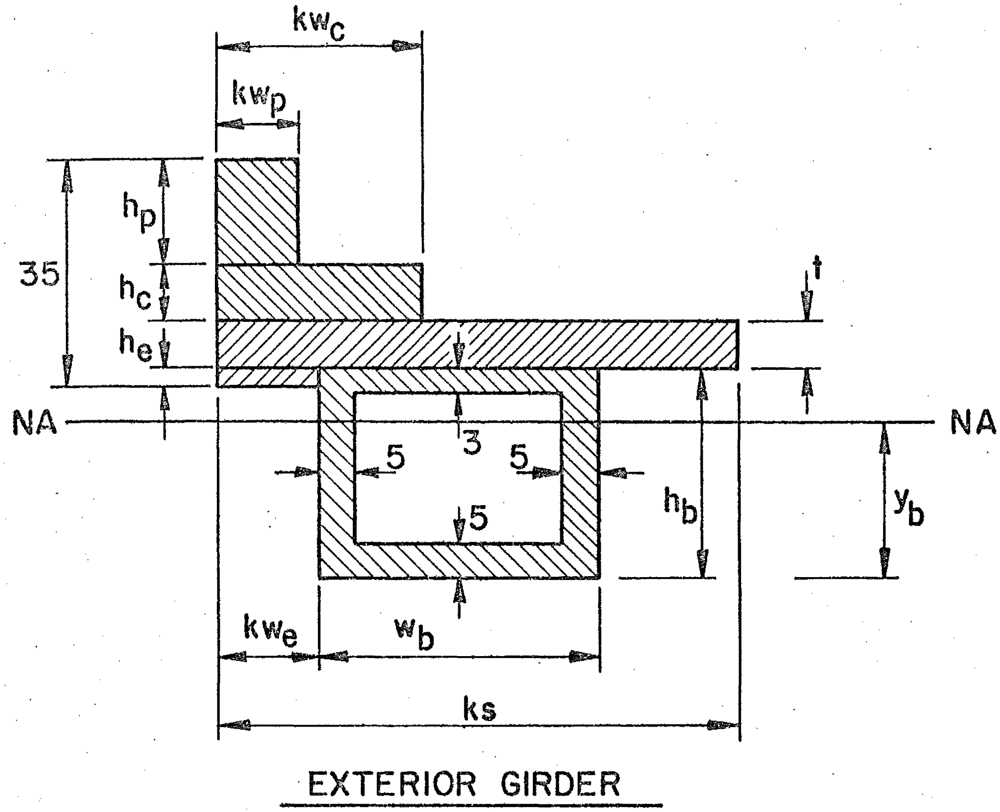


Fig. 36 Experimentally Developed Distribution Factors and Design Values: White Haven Bridge - Section N



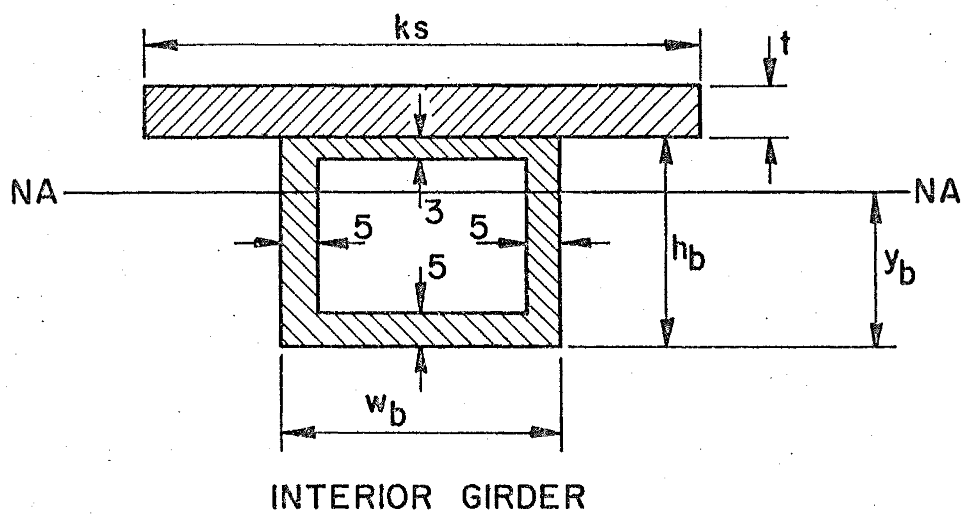


Fig. 37 Idealized Composite Cross-sections

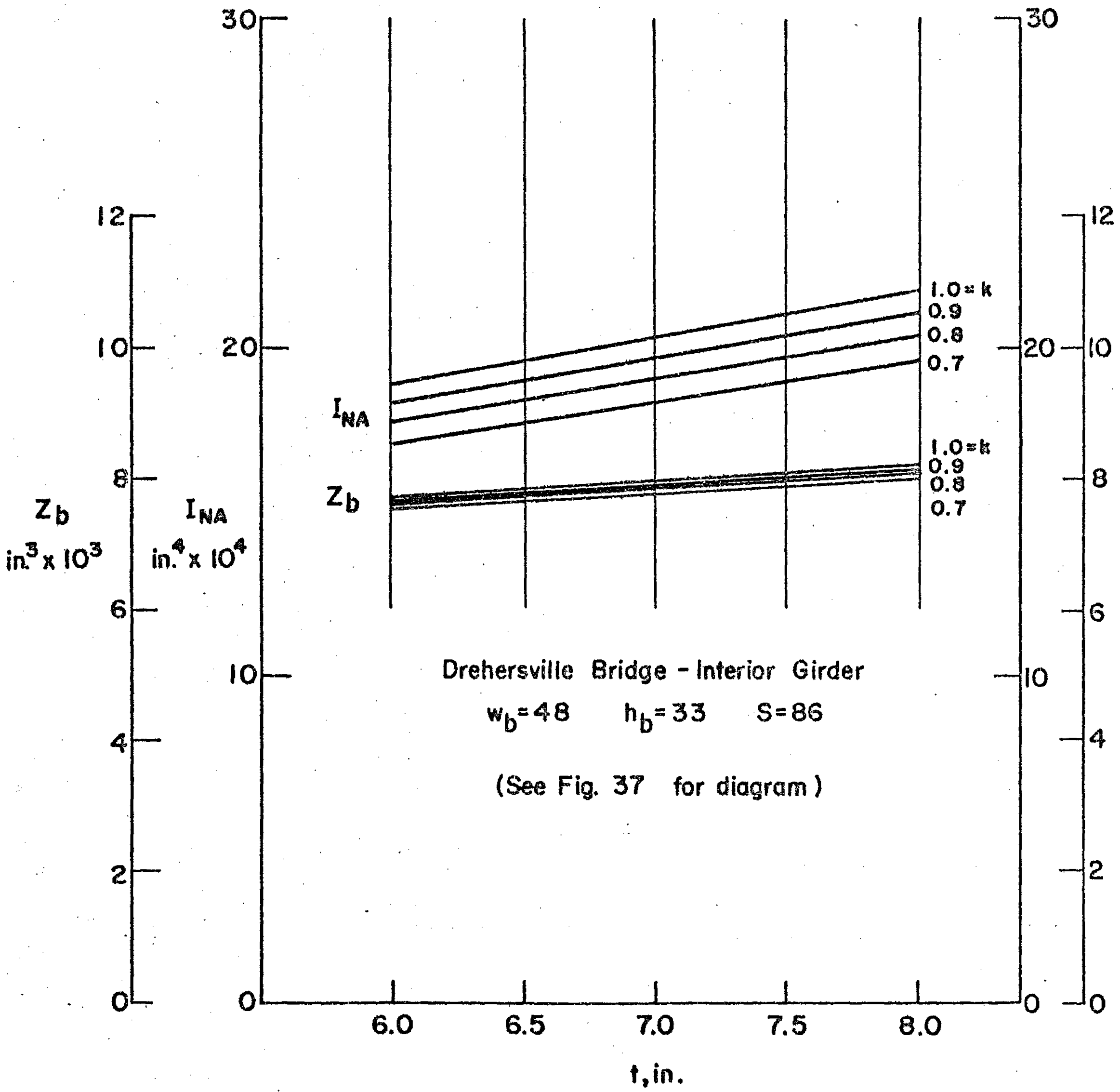


Fig. 38 Cross-sectional Properties - Interior Girder Drehersville Bridge

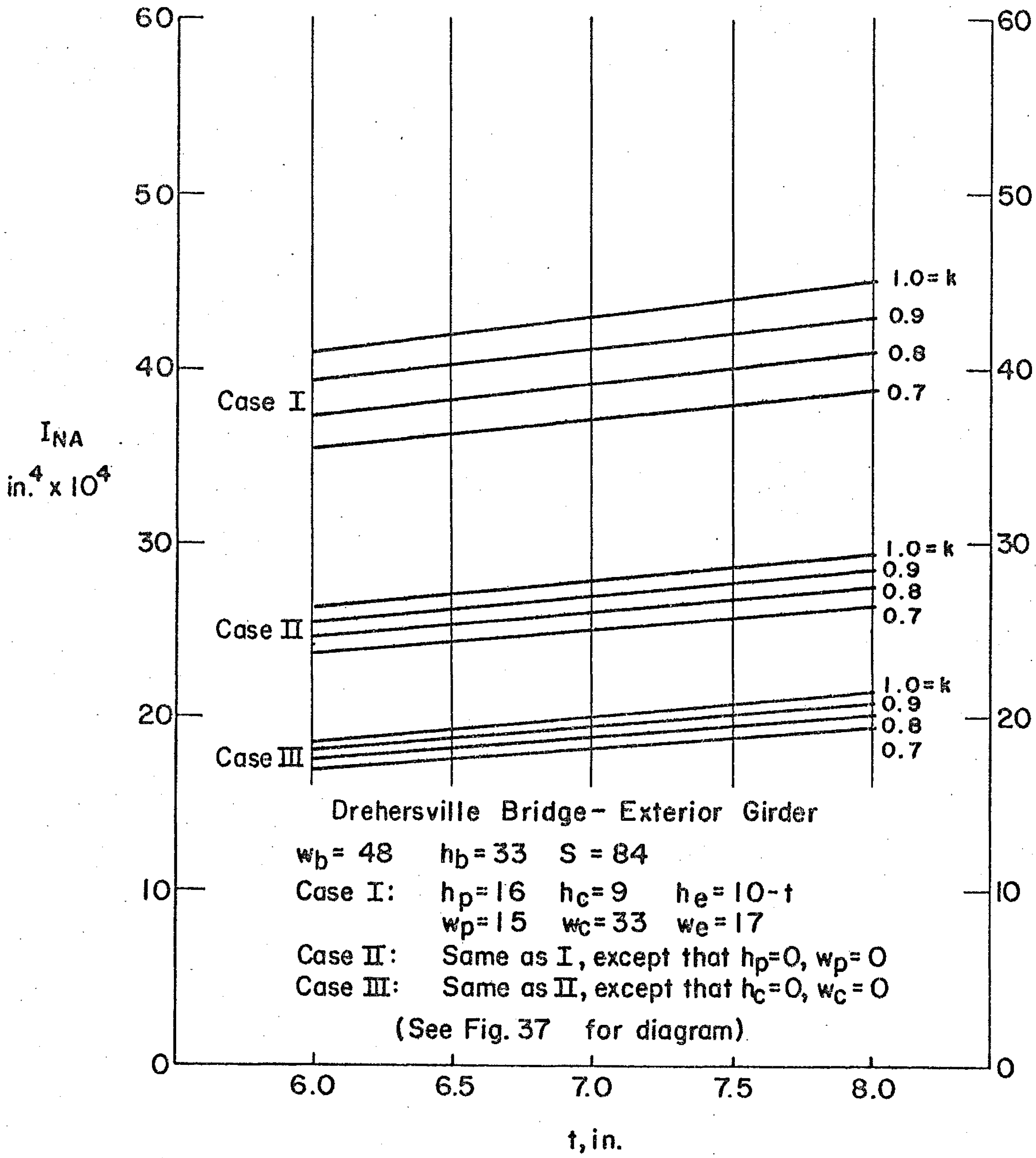


Fig. 39 Moment-of-Inertia - Exterior Girder Drehersville Bridge

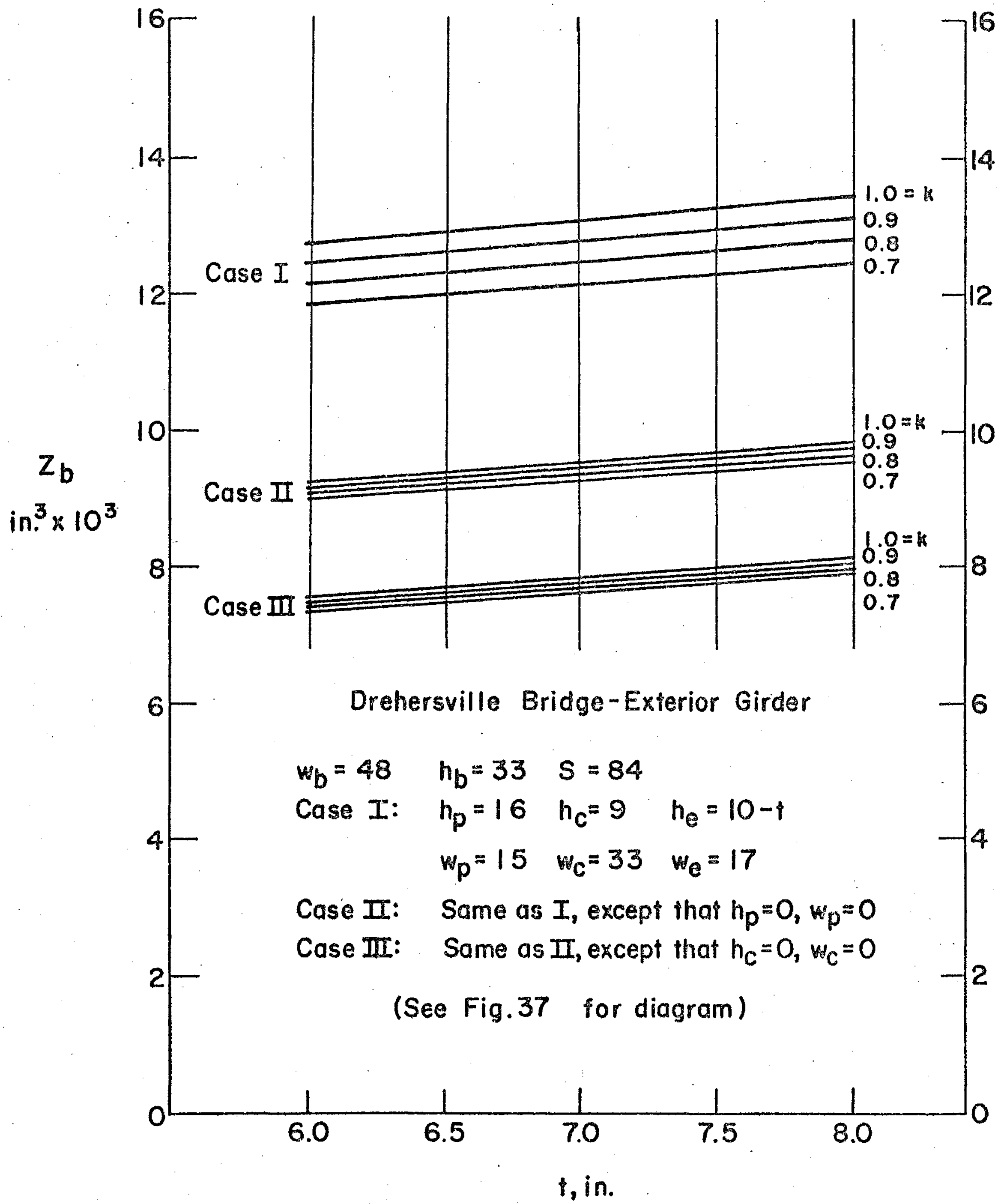


Fig. 40 Section Modulus - Exterior Girder
Drehersville Bridge

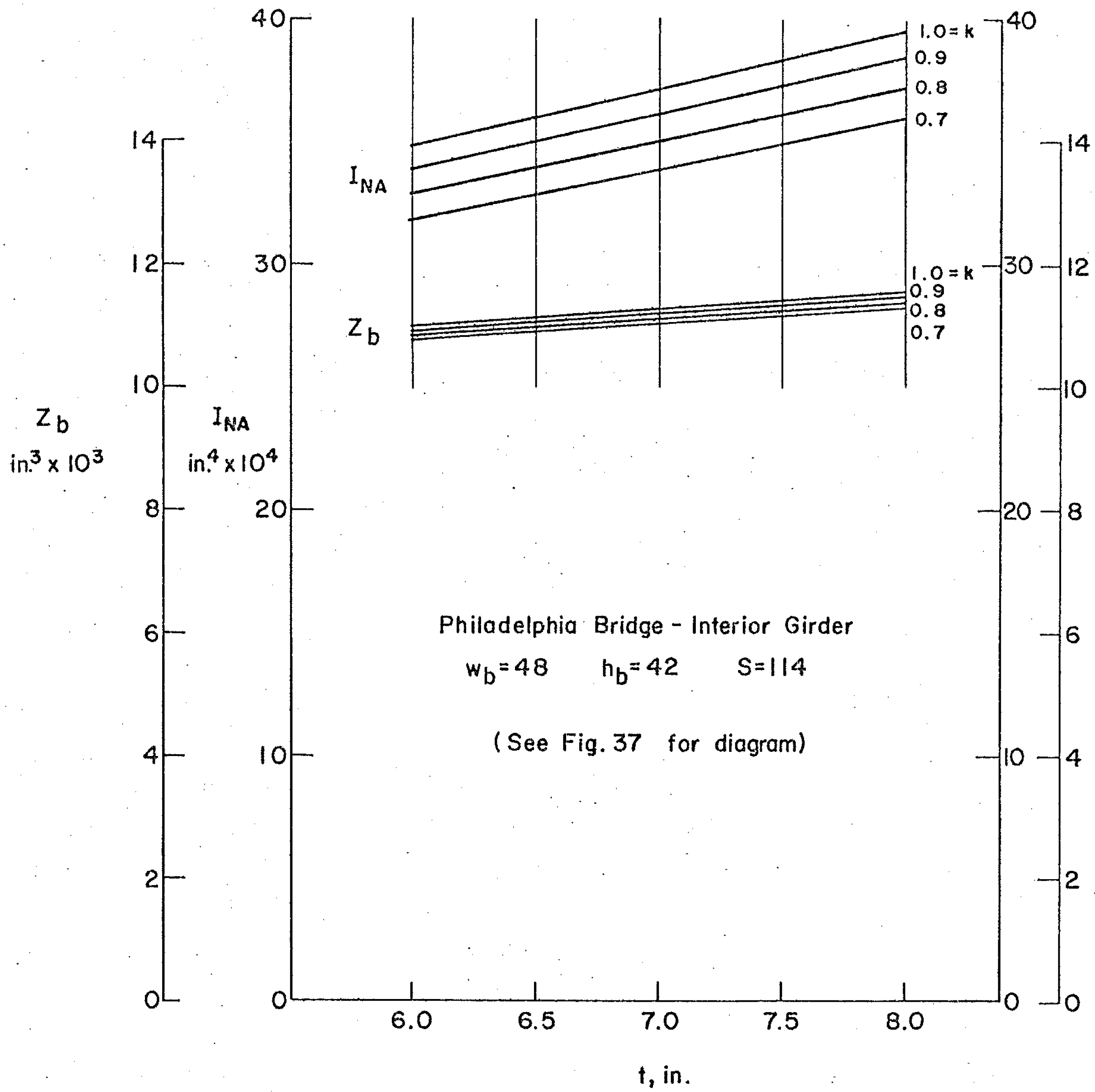


Fig. 41 Cross-sectional Properties - Interior Girder Philadelphia Bridge

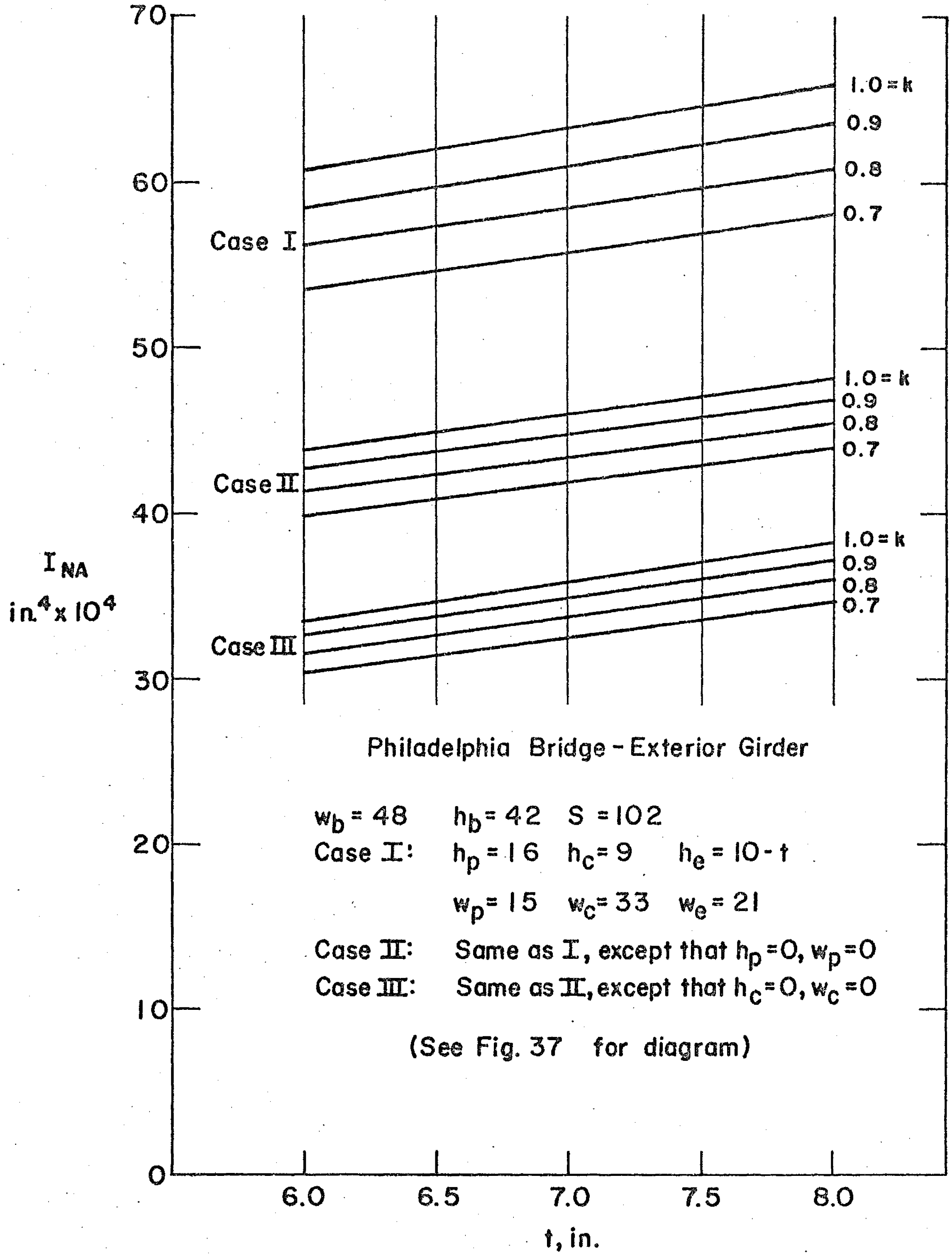


Fig. 42 Moment-of-Inertia - Exterior Girder
Philadelphia Bridge

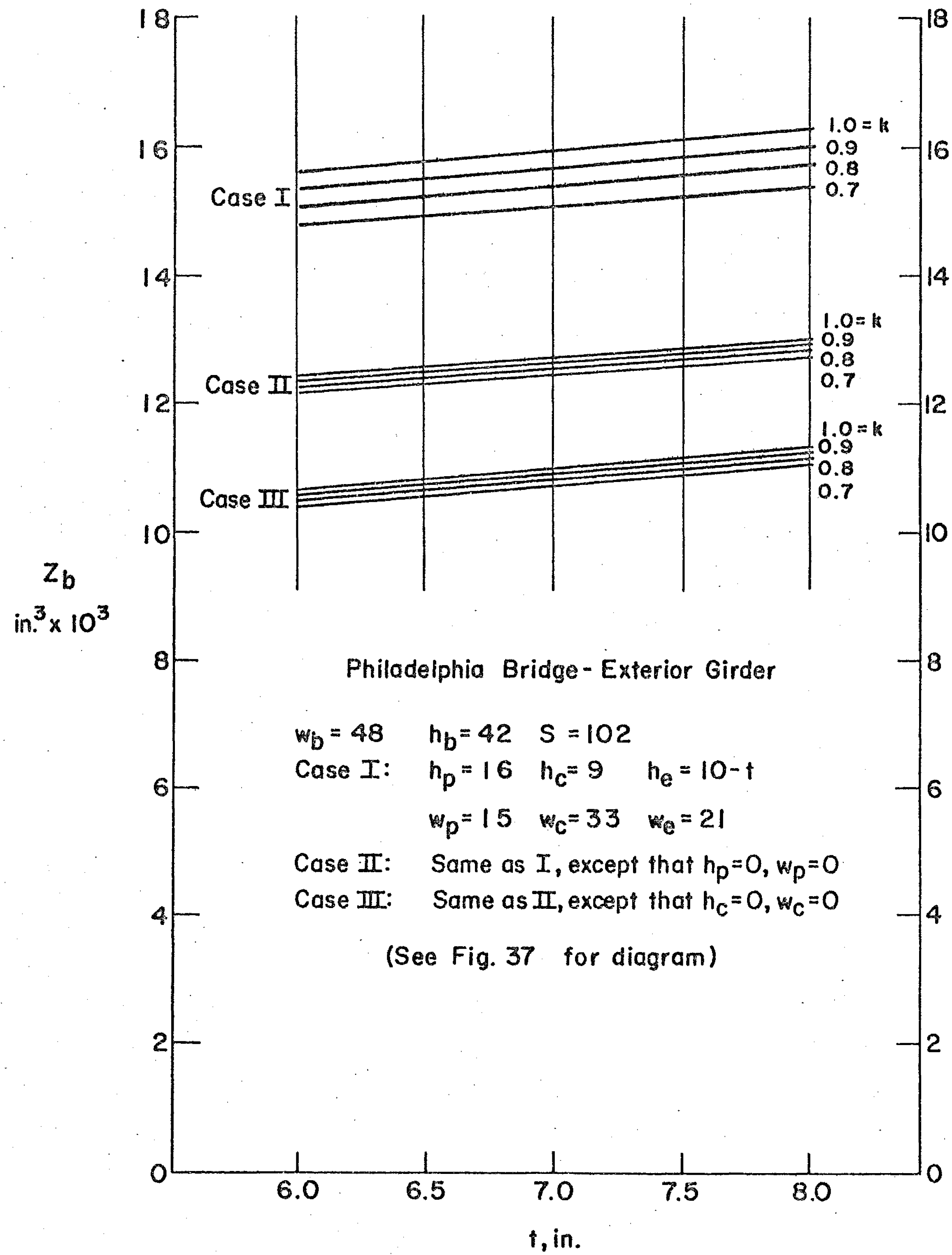


Fig. 43 Section Modulus - Exterior Girder Philadelphia Bridge

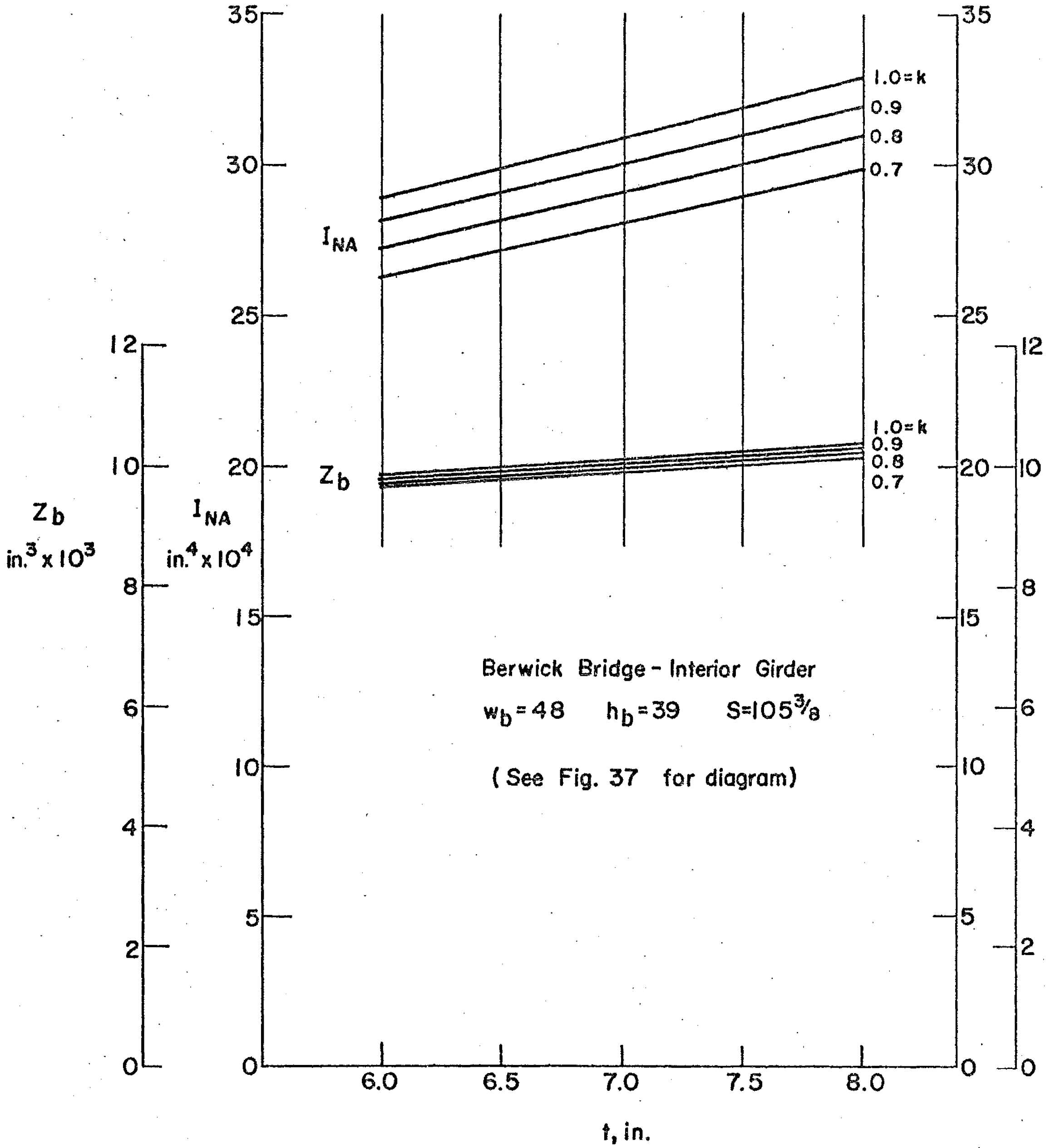


Fig. 44 Cross-sectional Properties - Interior Girder
Berwick Bridge

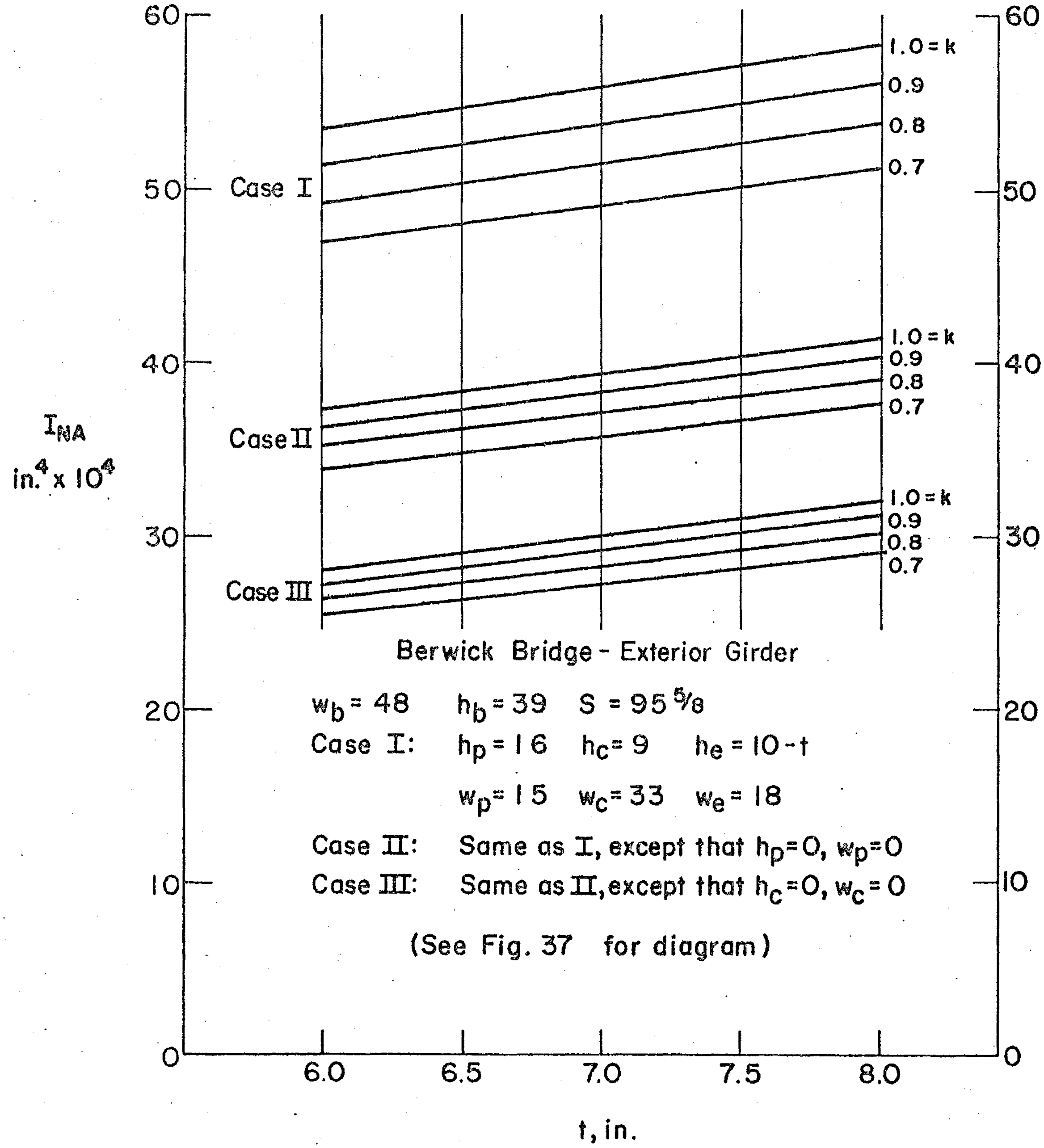


Fig. 45 Moment-of-Inertia - Exterior Girder
Berwick Bridge

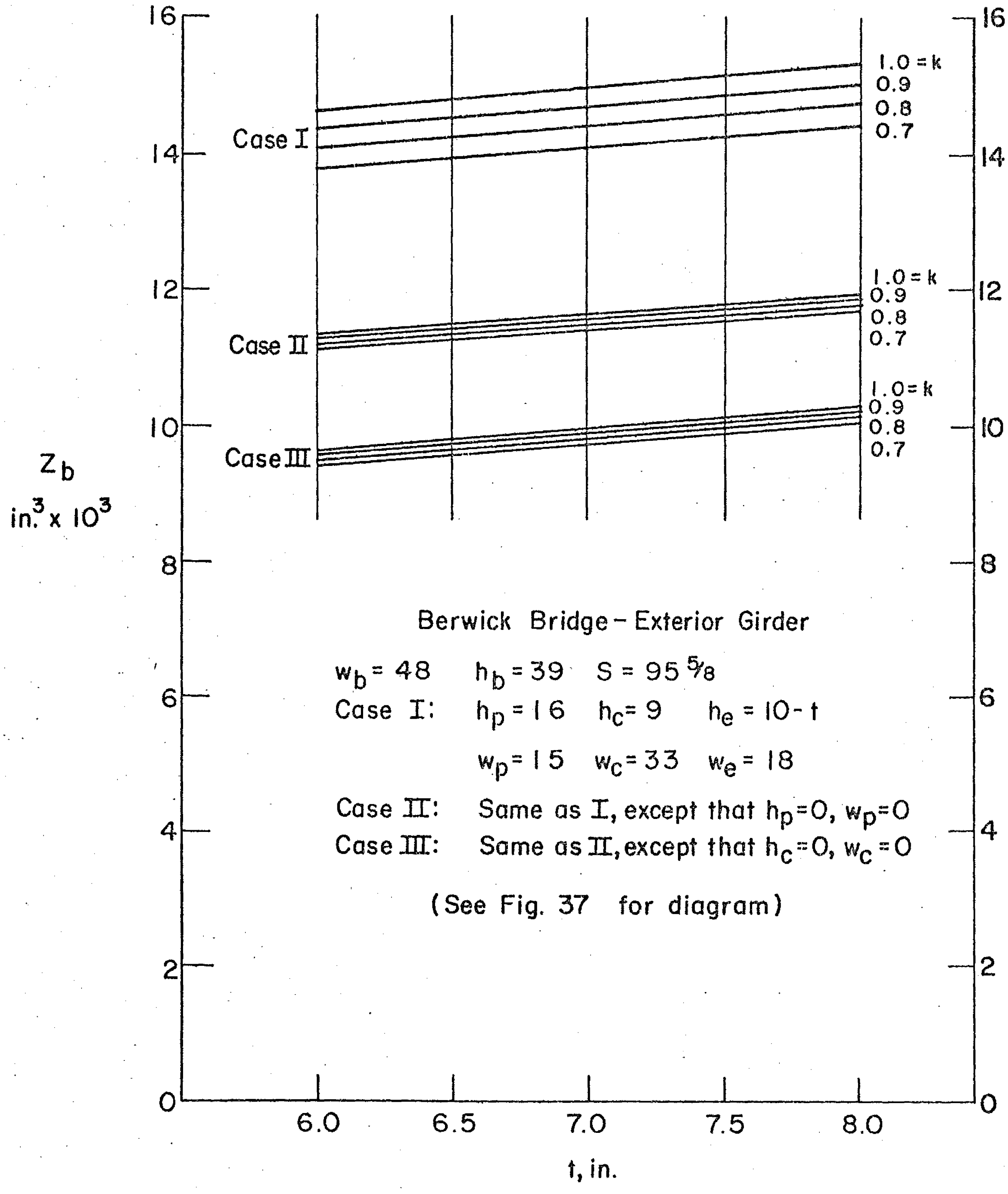


Fig. 46 Section Modulus - Exterior Girder
Berwick Bridge

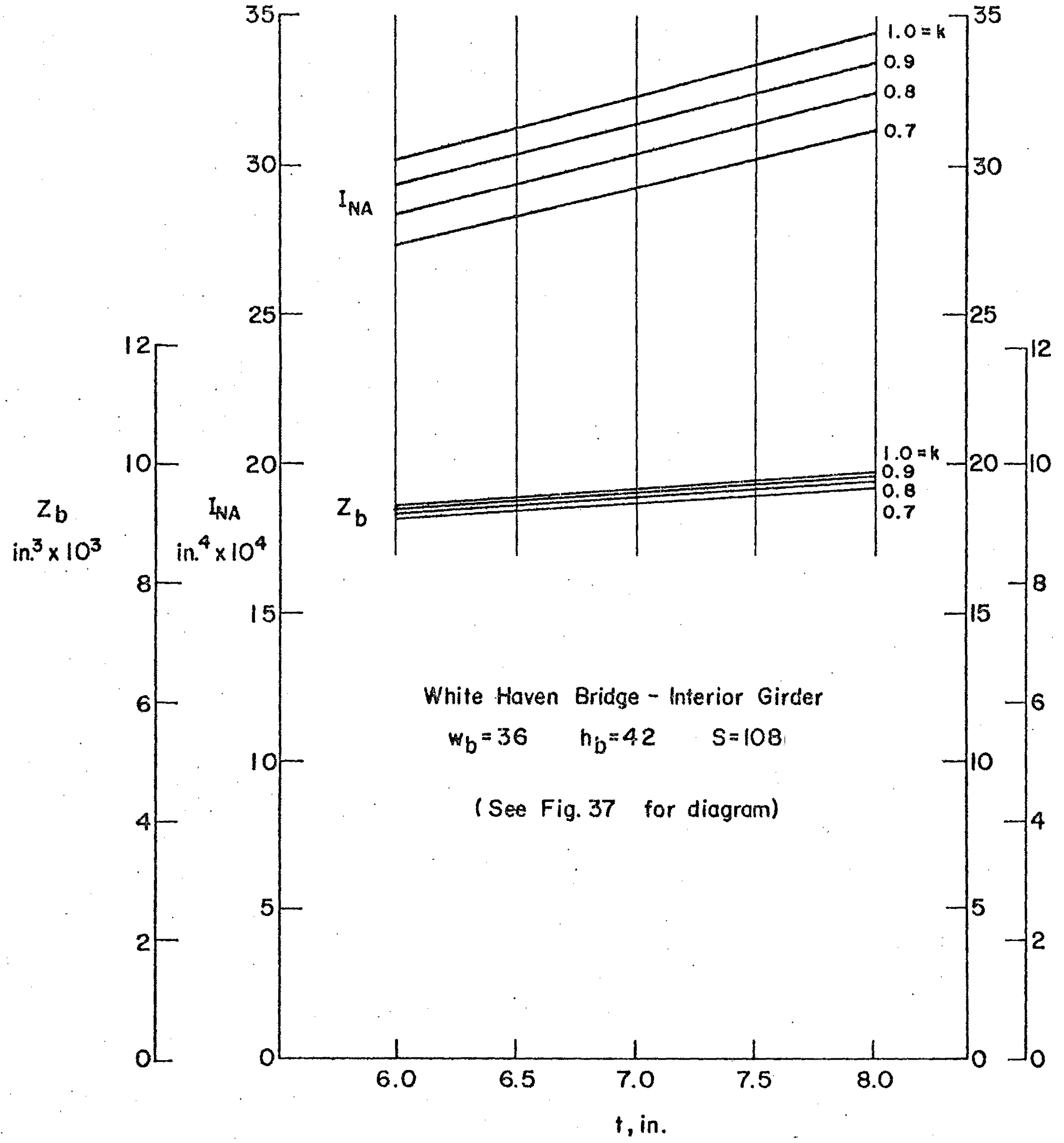


Fig. 47 Cross-sectional Properties - Interior Girder White Haven Bridge

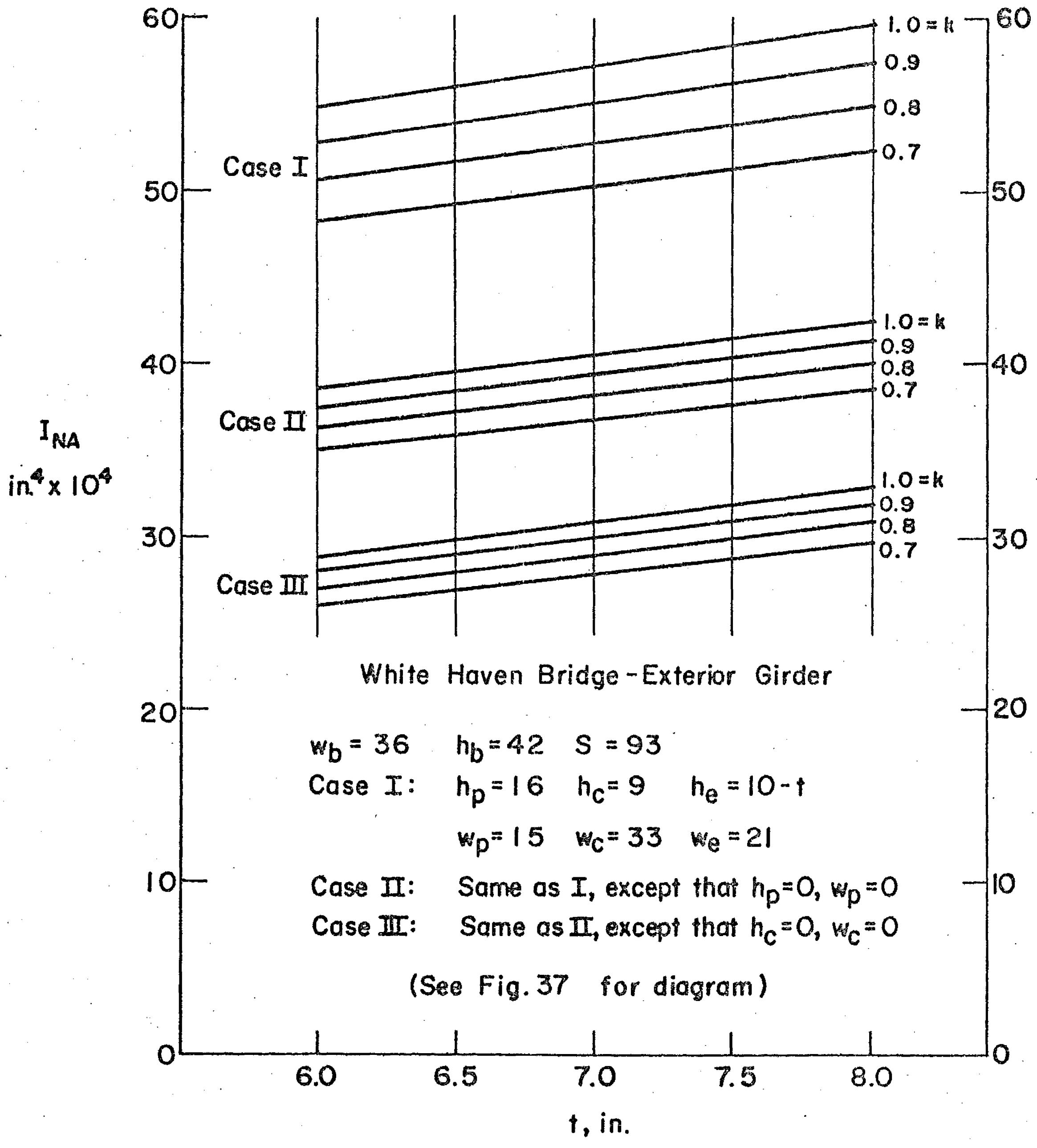


Fig. 48 Moment-of-Inertia - Exterior Girder
White Haven Bridge

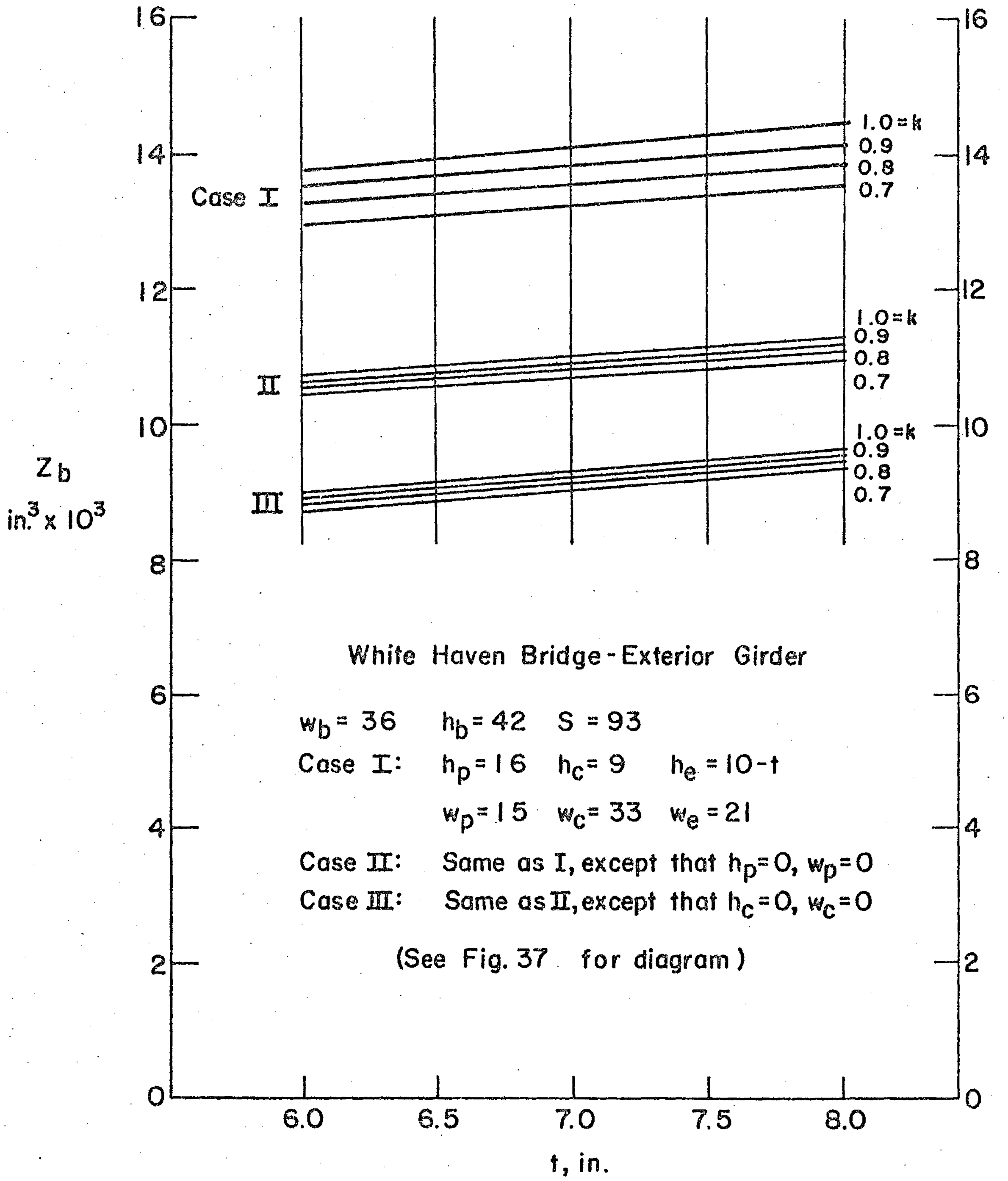


Fig. 49 Section Modulus - Exterior Girder
White Haven Bridge

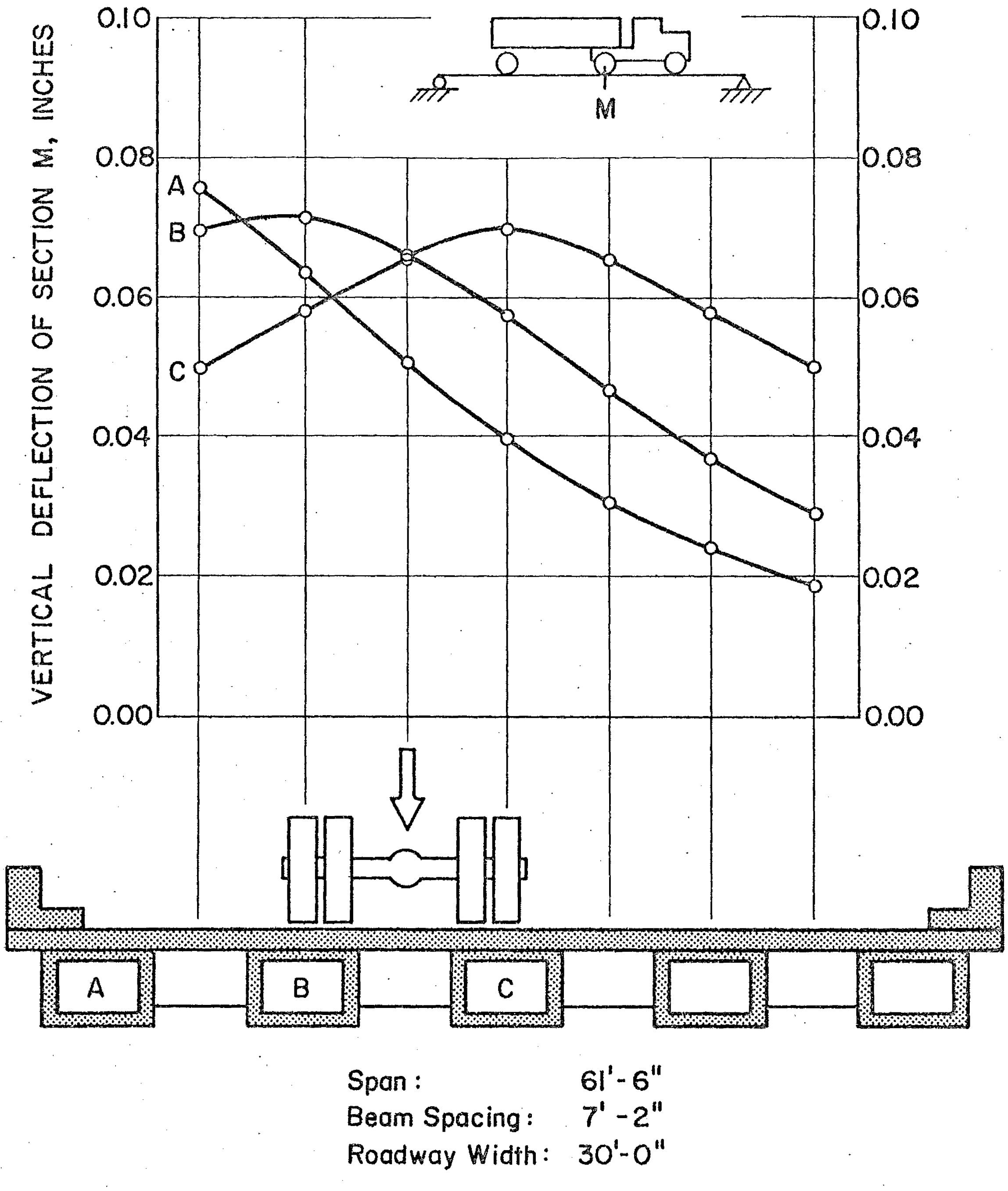


Fig. 50 Influence Lines for Vertical Deflections of Beams
Drehersville Bridge - Section M

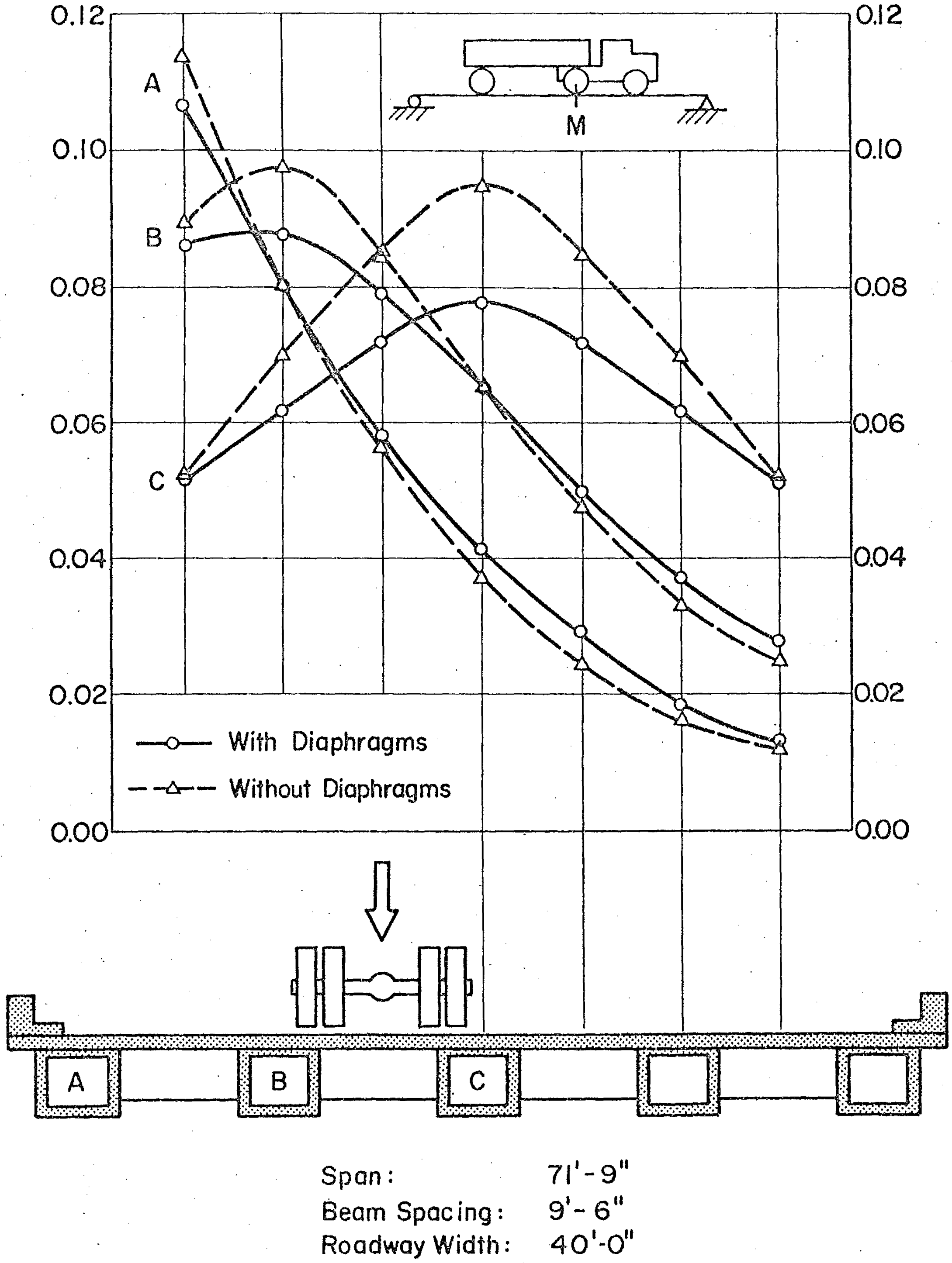


Fig. 51 Influence Lines for Vertical Deflections of Beams
Philadelphia Bridge - Section M

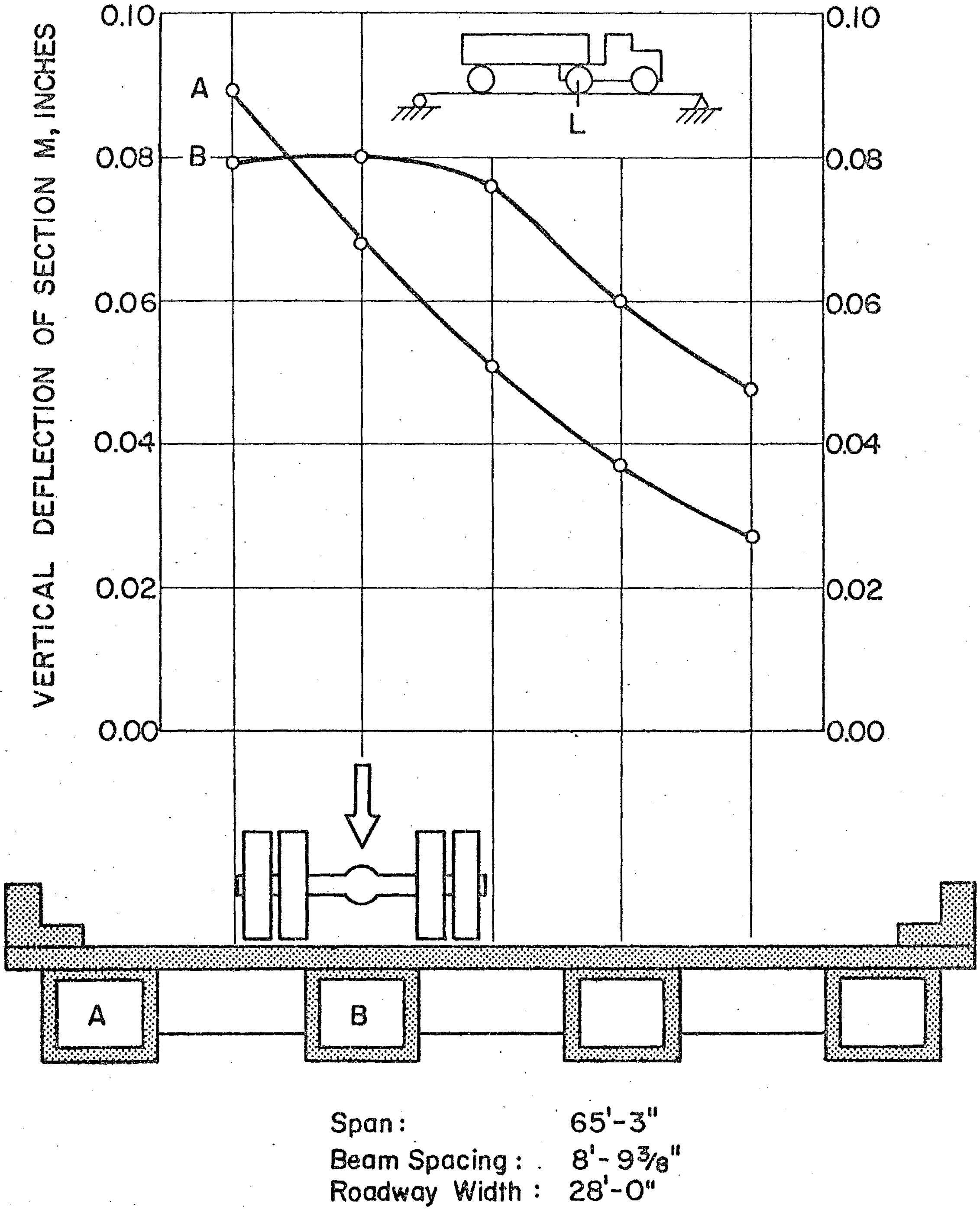
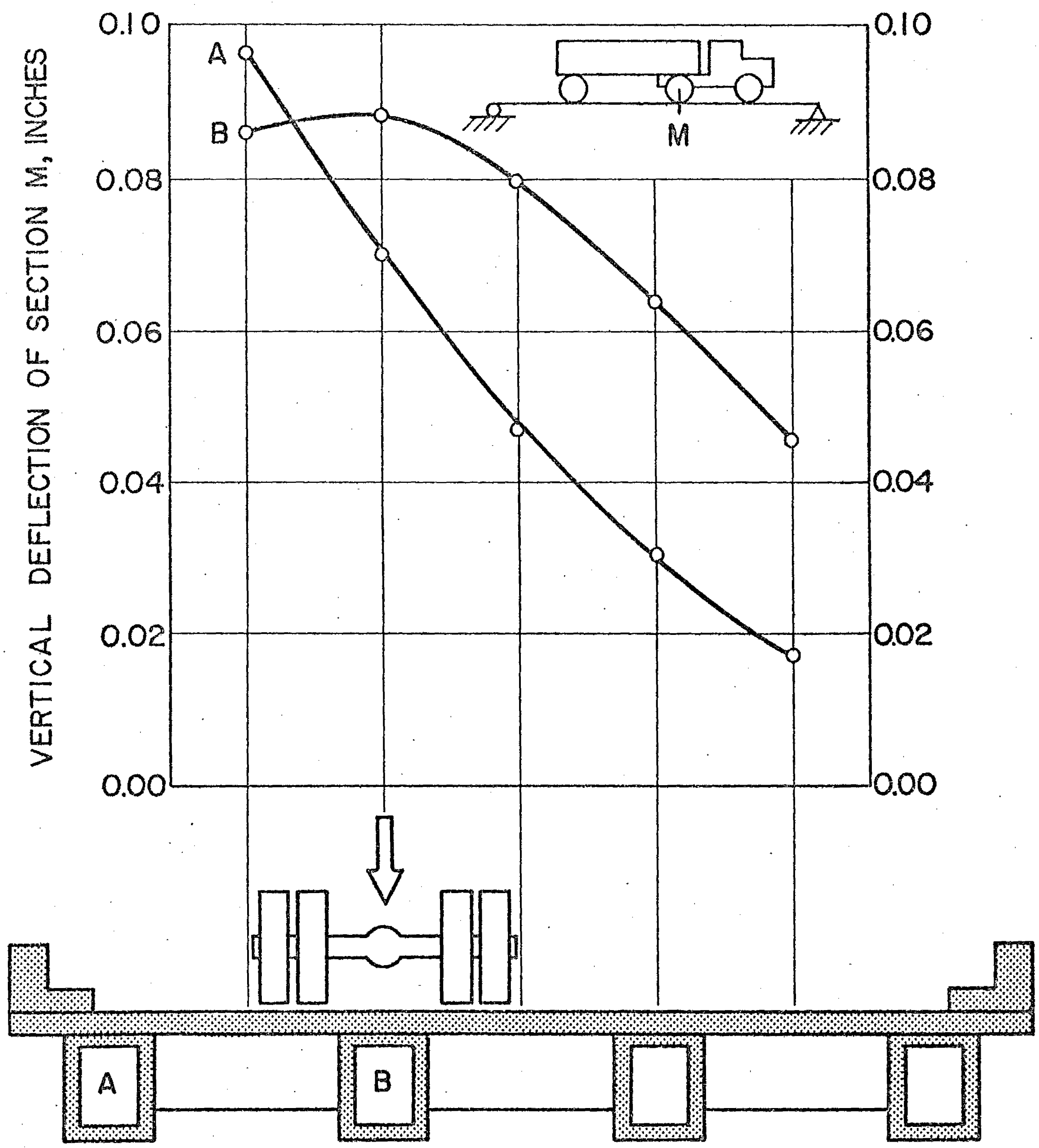
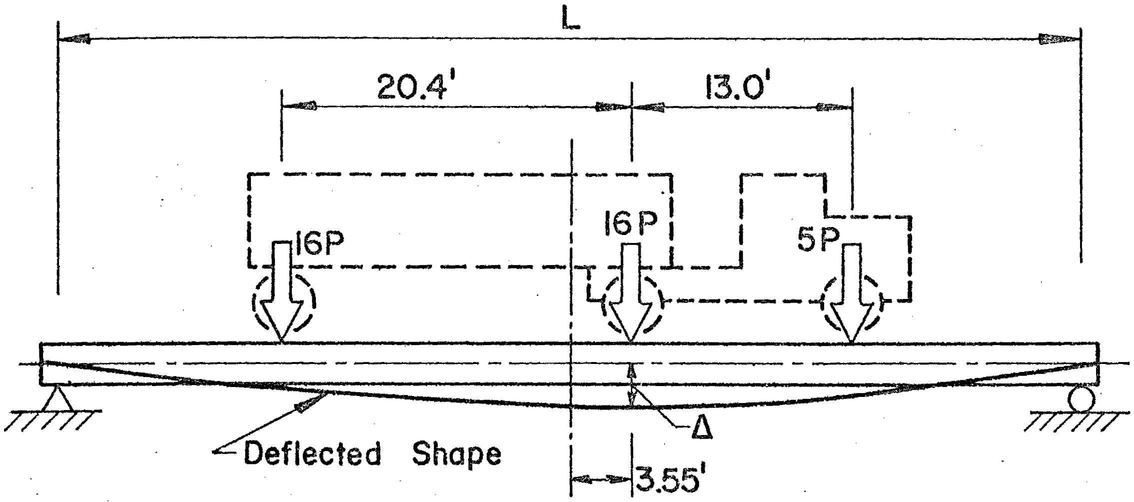


Fig. 52 Influence Lines for Vertical Deflections of Beams
Berwick Bridge - Section L



Span: 64'-8"
Beam Spacing : 9'-0"
Roadway Width 28'-0"

Fig. 53 Influence Lines for Vertical Deflections of Beams White Haven Bridge - Section M



$$\Delta = C \frac{PL^3}{EI}$$

Bridge	C	P kips	L ft.	E* ksi	I** in.^{4} x10^6	Δ in.
Drehersville	0.594	4	61.5	6800	1.33	0.106
Philadelphia	0.627	6	71.8	5400	2.19	0.203
Berwick	0.615	4	65.2	7300	1.58	0.102
White Haven	0.614	4	64.7	6000	1.60	0.120

* Measured in Flexural Investigation

** Assumed as Sum of Values for Individual Composite Beams t = 7.5 in., k = 0.7, Curb and Parapet Fully Effective

See Figures 37 to 49

Fig. 54 Idealized Bridge Deflections - Test Vehicle at Section of Maximum Moment

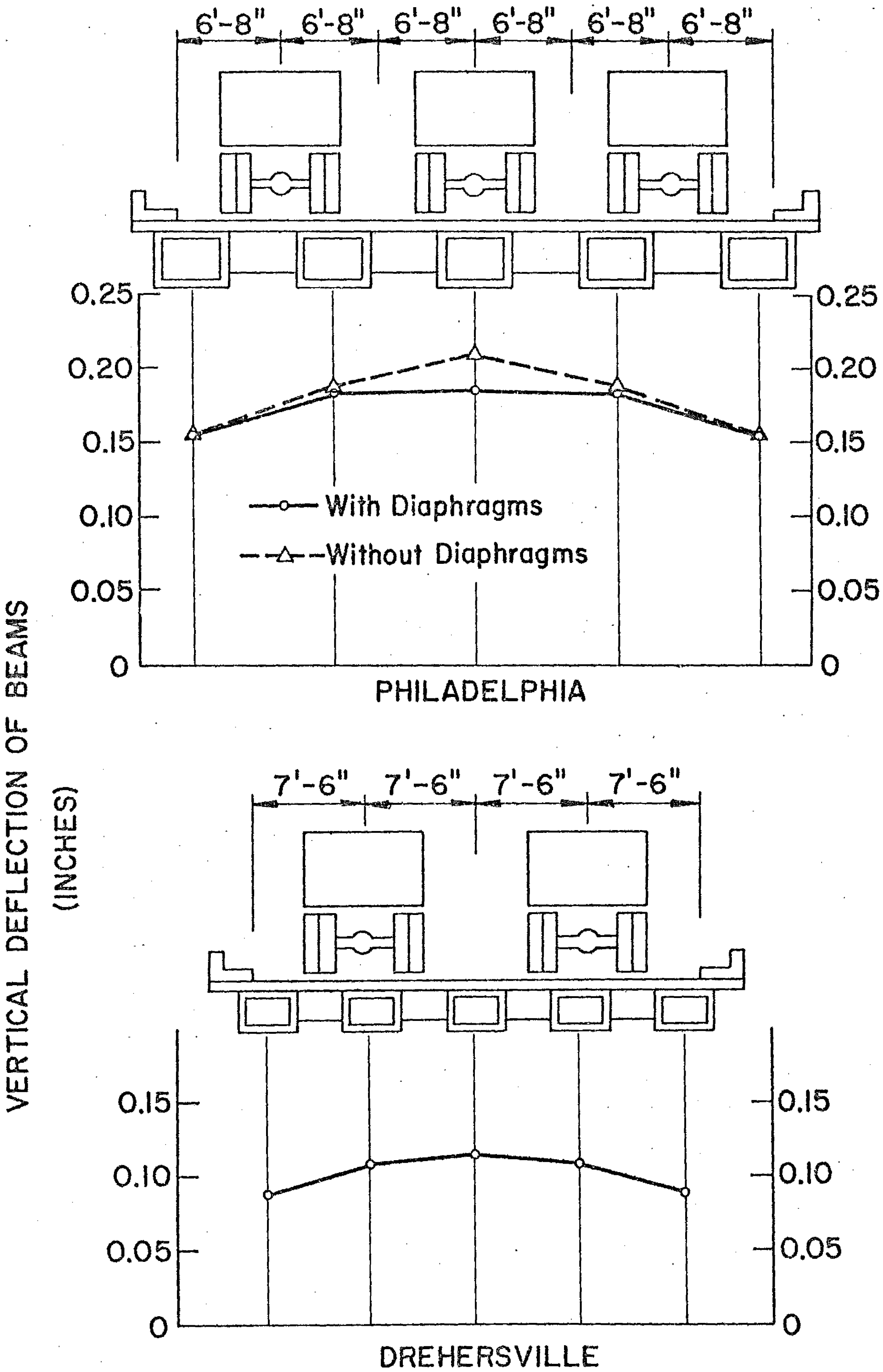


Fig. 55 Beam Deflection Profiles (All lanes loaded) Drehersville and Philadelphia Bridges

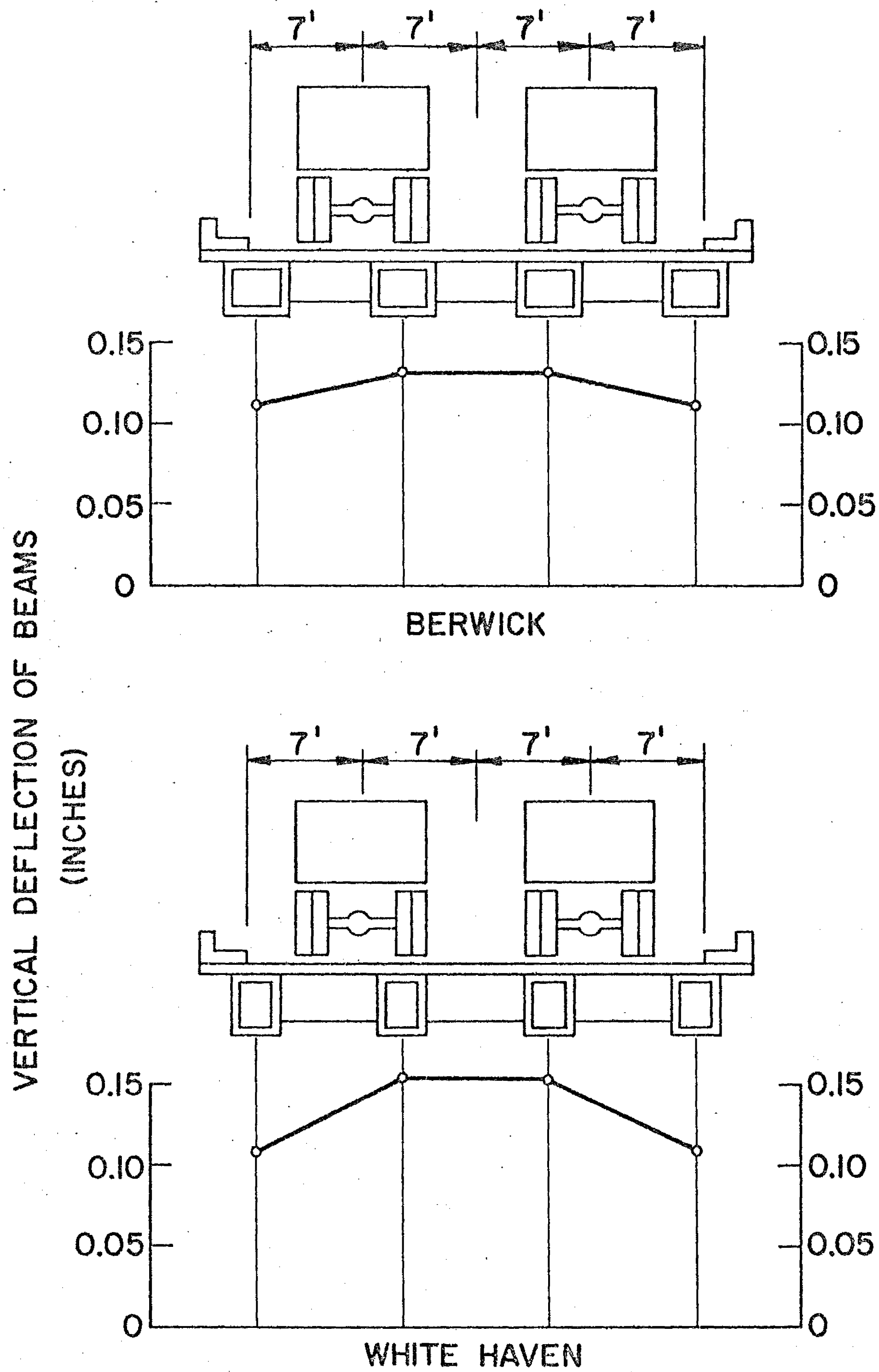


Fig. 56 Beam Deflection Profiles (All lanes loaded)
Berwick and White Haven Bridges

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